RIVER DISCHARGE

PREPARED FOR THE USE OF ENGINEERS AND STUDENTS

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PREFACE TO FOURTH EDITION.

Developments in the application of water to power, irrigation, and other engineering works during the last decade have been accompanied by such progress in the development of methods for collecting and using data concerning the flow of streams that the art of river-discharge measurement has attained a recognized position as a branch of hydraulics. The subject is now included in the regular courses of engineering schools and is admittedly essential to the work of the practicing engineer.

The first edition of River Discharge correlated the methods of collecting, analyzing, and using stream-flow records as developed at that time; the second and third editions were revised and expanded to include progress in the art; this new edition ha[illegible] contain the latest information on the subject.

The present revision has been practically limited to [illegible] the last part of Chapter V, which have been largely rewritten. Chapter VI, formerly entitled "Conditions affecting stream flow," has been expanded in scope to cover the field of hydrology as related to stream flow and the title changed accordingly.

The book has been prepared for the use of both student and engineer. Clearness and conciseness have been sought, and lengthy theoretical and mathematical discussions have been avoided.

J. C. H.
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WASHINGTON, D. C.,
August, 1916.

PREFACE TO FIRST EDITION.

With the rapid increase in the development of the water resources of the United States there has arisen among capitalists and engineers throughout the country a great demand for information in regard to the flow of streams. Although much has been written on the methods of measuring stream flow and the interpretation of the data, such information is widely scattered through periodicals and Government reports, many of which are out of print and therefore not easily accessible for use by either the student or the engineer. The short descriptions of stream gaging in text-books are indefinite in character, stating only general methods and giving but little information in regard to the details of field work or the conditions requisite for reliable records of river discharge.

Experience with the graduates of many of the best engineering schools in the country indicates that these men have generally had but little instruction in hydraulic field work or methods, and are practically helpless in attempting to carry on even the simplest hydrologic investigation
Correspondence with engineers in all sections o
they are not getting the maximum benefit from the available strea
gaging data, apparently on account of lack of understanding of the records.

In the preparation of this book there has been brought together from all available sources information in regard to the best practice in this work. Much new matter is also presented, especially the descriptions of the conditions necessary for good gaging stations at which measurements of discharge may be made either by weir, current meters, floats, or slope; the routine of the selection, establishment, and maintenance of gaging stations; the details of the field work of discharge measurements, and the office methods of computing the regimen of flow.

The authors hope and believe that the information here presented will be valuable both to the student and the engineer.

Acknowledgments are here made to the United States Geological Survey, the United States Weather Bureau, and the American Society of Civil Engineers, for use of cuts and other material; also to Messrs. J. C. Stevens, R. H. Bolster, G. M. Wood, F. W. Hanna, and E. C. Murphy for assistance and suggestions.

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By John C. Hoyt and Nathan C. Grover

CHAPTER I.

INTRODUCTION.

HISTORICAL SKETCH.

Practical acquaintance with and useful application of the general laws of flowing water date from the first century. In A. D. 98 Rome was supplied with water by nine aqueducts having an aggregate length of 250 miles and discharging 27,000,000 cubic feet a day. Yet hydraulics was not regarded as a science until about the fourteenth century and there was little advancement until the seventeenth century, when, owing to the influence of Galileo, more rapid progress was made. The principal investigations during the seventeenth and the first half of the eighteenth century were made by Castelli (1628), Torricelli (1643), Guglielmini (1700), Pitot (1730), and Bernouilli (1738), and the work done was mainly theoretical.

Active experimental hydraulic investigations were begun by Professor Michelotti in 1764, and from this time the modern school of hydraulics dates. Writings and investigations made prior to 1764 are now of comparatively little importance to the practicing engineer.[a]

In 1775 M. Chezy, the celebrated French engineer, developed the formula now known by his name, $V = c\sqrt{Rs}$, in which V = velocity and c = a coefficient combining the effects of roughness of the bed and all other conditions affecting velocity except the slope (s) and hydraulic radius (R), which equals the area of the cross-section of water divided by the wetted perimeter. This was the first algebraic expression of the law of moving water and has served as the basis of all subsequent slope formulas.

[a] A detailed review of early hydraulic studies is given in "Physics and Hydraulics of the Mississippi," by Humphreys and Abbot.

In the United States attention was first given to the flow of water in open channels between 1840 and 1850, in work on the Mississippi River and its tributaries. In 1850 Humphreys and Abbot started their extensive investigations on that river, and in about the same year Charles Ellet used gage heights and a rating curve based on discharge measurements to determine the daily discharge of Ohio River at Wheeling.[a] In 1855 Francis published the results of his investigations made at Lowell, Mass., in which he developed his formula for flow over weirs. In 1870 Ellis, in his work on the Connecticut River, added much valuable data. It was not until 1888, when the United States Geological Survey began to collect data in regard to the water supply of the country at large, that the general applicability of hydraulic laws was investigated and methods were developed for determining the regimen or the distribution of flow.

In starting the hydrographic work of the Survey, Major J. W. Powell, then Director, stated:[b]

> It will be necessary to gage a certain number of representative streams at all seasons of the year, so as to ascertain their total discharge and its seasonal distribution, and also to gage a greater number of streams at certain seasons determined [illegible]l.

[illegible]ing with this object, the Survey developed methods for universal stream gaging and collected data in regard to the flow of streams in all sections of the United States, which are now extensively used by engineers in enterprises involving the use of water. In all this work the Survey has contended that, inasmuch as the flow of a stream is constantly changing, systematic records showing the distribution of flow over several consecutive years are more valuable for nearly all uses than many broken records covering short periods of time.

SCOPE OF DISCUSSION.

The hydraulic engineer is interested in water from the time it reaches the earth in the form of rain or snow until it returns again to the atmosphere in the form of an invisible vapor. Of the water which falls upon the earth, a portion immediately returns to the atmosphere; a portion soaks into the earth, reappearing in vegetation or as surface water, or remaining below in small amount as permanent ground water; and another portion stays for a time on the surface of the earth, in streams, ponds, lakes, or oceans. A knowledge of the phenomena that pertain to these changes in conditions and of the physical and chemical prop-

[a] The Mississippi and Ohio rivers, Lippencott, Grambo & Co., 1853.

[b] Tenth Ann. Report, U. S. Geol. Survey [illegible] p. 8.

erties of the water itself constitutes the science of hydrology. Every feature of this great science is of direct value in the economic development of the country, but probably none is of greater importance than a knowledge of the discharge of surface streams and of the conditions that affect its magnitude and variations—knowledge that is prerequisite for preliminary as well as final plans for the construction and successful operation of works utilizing the water in surface streams. Among the hydrologic data necessary either for the design or operation of such works records of daily discharge are the most important. Isolated observations of stage or discharge are of little value unless made at stages that are known to be extreme, and even then the record of the duration is equal in importance to that of the magnitude of the flow. This discussion of surface flow is arranged under the following heads:

Instruments and equipment.
Velocity-area stations.
Weir stations.
Discussion and use of data.
Hydrology as related to stream flow.

OUTLINE OF METHODS.

The discharge of a stream is the quantity of water flowing past a given section in a unit of time and is expressed in various units, among which the second-foot is the most common. This term is an abbreviation for cubic foot per second, which is equivalent to the quantity of water flowing in a stream 1 foot wide, 1 foot deep, at a velocity of 1 foot per second. The determination of the discharge is termed "discharge measurement." The discharge may be obtained as the product of two factors—(1) the area of cross-section, which depends on the shape and dimensions of the bed and banks and on the stage; (2) the velocity, which depends on the surface slope, the roughness of the bed and banks, the hydraulic radius, and the conditions along the channel of the stream. In general these factors are controlled by the stage. Therefore the discharge may be considered as a function of the stage.

By means of this general law it is possible, from discharge measurements covering the range of stage, to construct a rating curve and table from which, the mean daily stage of the stream being known, the daily discharge can be taken. Points at which discharge measurements are made and records of the daily fluctuations of stage are kept for determining the daily flow are termed "gaging stations." These stations may be grouped in two classes, one comprising those where measure-

ments are made by the velocity-area method, which consists in measuring the velocity of the current and the area of the cross-section; the other comprising those where measurements are made by the weir method, in which the discharge is obtained by measuring the head on a weir and using a weir formula.

The selection of a gaging station, the equipment, and the method to be used in determining the discharge depend on many factors and are accomplished in various ways. Among the principal factors are the use for which the records are to be collected, the funds available, the period of time over which the observations are to be extended, and the conditions of the stream to be measured, as explained in the following pages.

CHAPTER II.

INSTRUMENTS AND EQUIPMENT.[a]

The establishment and maintenance of gaging stations for obtaining records of discharge of rivers and other hydrologic data require the use of certain instruments and equipment. These may consist of:

1. Instruments for determining the velocity and other factors of the discharge measurement.
2. Gages and bench marks for determining stage relative to a fixed datum.
3. Structures from which discharge measurements are made and the appurtenances thereto.
4. Structures to produce artificial control and regulate the relation between stage and discharge.
5. Instruments for determining climatological data.

INSTRUMENTS FOR DETERMINING VELOCITY.

Two principal types of instruments are used for measuring the velocity of flowing water—floats, which measure the velocity directly, and current meters, by which the velocity is obtained indirectly from observations of the number of revolutions of the wheel. Another instrument sometimes used for measuring velocity is the Pitot tube, but it is not practicable to use this tube for the work discussed in this book.

FLOATS.

Floats are utilized for the direct measurement of the velocity of streams. Those in common use are surface, subsurface, and tube or rod floats.

Surface floats.—A corked bottle with a flag in the top and a weight in the bottom makes a very satisfactory surface float, as it is but little affected by the wind. In flood measurements good results can be obtained by observing the velocity of débris or of floating cakes of ice. In all surface-float measurements coefficients must be used to reduce observed velocities to the mean velocity.

Subsurface floats.—The subsurface float (Fig. 1) is designed to measure velocities below the surface and may be made to float at any depth. By

[a] See Water-Supply Paper No. 371, U. S. Geol. Survey.

arranging the submerged float at the depth of mean velocity it may be utilized in observing mean velocity directly. Allowance must be made, however, for the accelerating effect of the attached line and surface float.

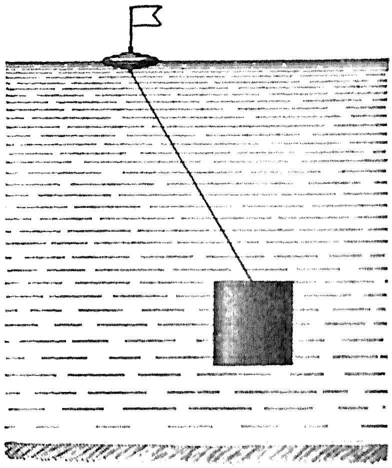


FIG. 1.—Subsurface Float.

Tube or rod floats.—The tube or rod float is designed also to measure directly the mean velocity in a vertical. It is generally a cylinder of tin, about 2½ inches in diameter, weighted at its lower end and plugged with wood or cork at its top. Small extra weight to make it float at the exact depth desired may readily be added by admitting water or by putting in shot. The tube should be graduated, and alternate feet painted black and red in order that the depth of flotation may be readily observed.

A number of tubes of different lengths are necessary for measuring the velocity at different depths in an ordinary cross-section. A float of this type is consequently best adapted for use in artificial channels, in which the depth is nearly uniform, as natural channels are generally too rough and too variable to permit its satisfactory use.

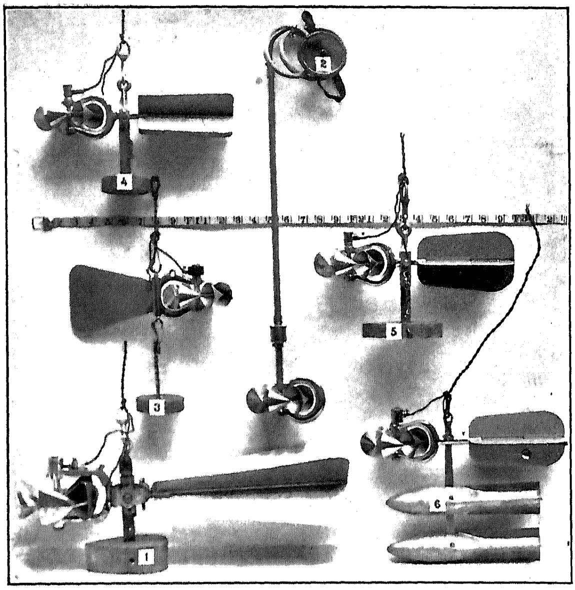
Although designed to measure directly the mean velocity in a vertical, the tube can not be made to float in contact with the bed of the stream, and consequently it does not receive the effect of the slowest moving water. The rougher the bed the greater the error in this respect. A factor less than unity is therefore necessary to reduce the observed velocity to the mean.

CURRENT METERS.[a]

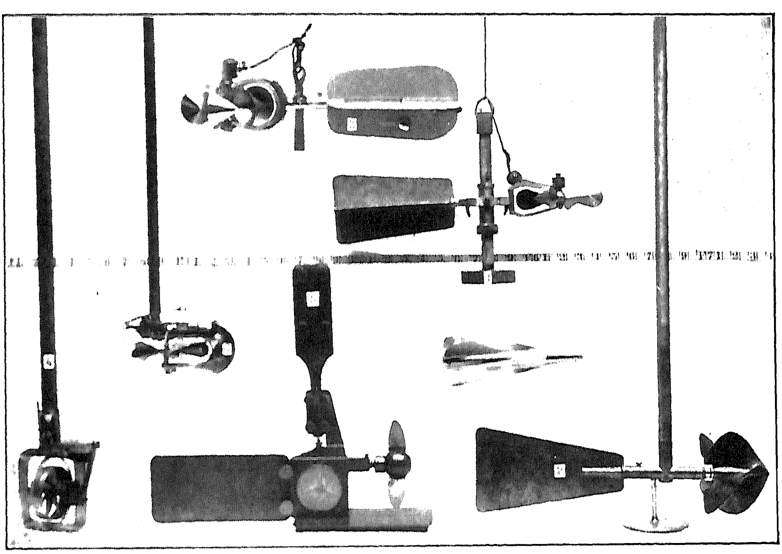
A current meter for measuring the velocity of flowing water comprises two essential parts: (*a*) a wheel arranged so that when suspended in flowing water the pressure of the water against it causes it to revolve; (*b*) a device for recording or indicating the number of revolutions of this wheel. The relation between the velocity of the moving water and the revolutions of the wheel is determined by rating each meter.

The earliest type of meter was the float wheel, which was used by Borda

[a] Transactions American Society of Civil Engineers, Paper No. [illegible]



A. VARIOUS FORMS OF THE PRICE METER.



B. TYPES OF METERS EXPERIMENTED WITH BY UNITED STATES GEOLOGICAL SURVEY.

and Dupuit in the latter part of the eighteenth century, and was practicable only for measuring velocities at the surface. About 1790, Woltmann modified this wheel so that it could be used beneath the surface, the number of revolutions being recorded by a gear mechanism, which was started and stopped at the beginning and end of a run by a catch operated by a cord. It was necessary, however, to lift the meter out of the water in order to read it. LaPointe arranged the recording apparatus above the surface by connecting the axle with a vertical rod and beveled gear. Baumgarten, Saxton, Brewster, Laignel, and others made various modifications of the instrument. Prior to the invention of an electric device for recording or indicating the number of revolutions of the wheel, the meter was of limited use because of its lack of adaptability to varying conditions and because of difficulties with the operation of the recording mechanism.

In America current meters were earliest used in connection with the investigations of the Mississippi, started in 1850 by Humphreys and Abbot,[a] in which the ship's log and the Saxton meter were used to a small extent and with little success.

About 1860, the late D. Farrand Henry, M. Am. Soc. C. E., Assistant, United States Lake Survey, invented for use with the current meter an electrical recorder,[b] which eliminated the serious difficulties peculiar to the mechanical recorder, and made feasible the further development of the meter.

The first extended and successful series of measurements with the current meter in the United States was made on Connecticut River by the late T. G. Ellis, M. Am. Soc. C. E., in connection with studies begun in 1871.[c] General Ellis started his work with the Woltmann meter, equipped with an electrical recording device, but later used an electrical recording meter devised by himself. The results obtained by these measurements have had an important effect on the development of stream-gaging instruments.

The earliest American patents for current meters were taken out in 1851. There are now on file in the Patent Office, classified under ship's logs, more than fifty patents for devices for measuring the velocity of water. Many unpatented devices have also been constructed. The only meters which have had much general use, however, are those devised by Price, Haskell, Fteley, and Ellis or modifications of these types (Pl. I, B, Nos. 3-2-4 and 1).

Each of the various meters has first been developed to meet the require-

[a] Report upon the Physics and Hydraulics of the Mississippi River, 1861.
[b] Journal of the Franklin Institute, Vol. XCII, 1871.
[c] Report, Chief of Engineers, U. S. Army, 1878, Part I.

ments of some special condition, and, until recently, the use of all has been confined to special hydrologic investigations in connection with some public work, municipal, State, or Federal. The present widespread interest in the value and use of water has created such a demand for records of the discharge of streams that the current meter is now in general use, and has become an essential part of the equipment of every engineer engaged in hydraulic work.

In 1888 the United States Geological Survey began the gaging of streams of all sizes and in all sections of the country. These streams presented an infinite variety in combination of range in depth, width, and velocity. No adequate meter or methods had been developed for work of this varied nature. Furthermore, elaborate equipment and methods were out of question on account of the limited funds. It was necessary to devise or adapt a current meter which could be readily carried in the field and operated by one man, either from a bridge, boat, cable and car, or by wading.

After experimenting with various types (Pl. I, B) the engineers of the Survey developed a meter combining certain essential features of the Price acoustic and the large Price electric meter[a] (Pl. I, A, Nos. 1 and 2.) This is known as the small Price meter, and has since been in general use in the Survey work. Modifications in its construction have been made from time to time until now it represents the ideas of many engineers, resulting from the experience of more than twenty years in stream gaging.

The methods developed by the Survey engineers are also believed to represent the best practice in this line of work. The Survey's data are now used extensively in all hydraulic development in the United States. Its methods have been accepted as standard in this country and have been adopted in similar work by many engineers in all parts of the world.

GENERAL FEATURES OF CURRENT METERS.

Current meters may be divided into two general classes: direct action and differential action, the division depending on whether the water, in revolving the wheel, does or does not exert a force which tends to retard the motion of the wheel.

The wheel of the direct-action meter consists of flat or warped-surface vanes set on a horizontal axis, which are caused to revolve by the direct pressure of the water against them. Each vane receives the water pressure in the same way as all of the others. The principal types of direct-action meters are the Haskell and Fteley (Pl. I, B, Nos. 2 and 4).

The wheel of the differential meter consists of a vertical axis carrying a series of cups which are revolved by the water pressure on the concave

[a] Manufactured and sold by W. & L. E. Gurley, Troy, N. Y.

side of the cups and are retarded by the lesser pressure on the convex side. The principal types of differential meters are the Price and the Ellis (Pl. I, B, Nos. 3 and 1).

The essentials for a good current meter are: (*a*) simplicity and lightness of construction, with no delicate parts which easily get out of order; (*b*) simplicity in operation, including its preparation for use under any conditions, and its dismantling, cleaning, and boxing after use; (*c*) a small area of resistance to the action of the water; (*d*) a simple and effective device for indicating the number of revolutions of the wheel; and (*e*) adaptability for use under all conditions.

The small Price meter is the only one fully described herein. The discussion on the care and use of current meters is, however, generally applicable to any type.

DESCRIPTION OF THE SMALL PRICE CURRENT METER AND EQUIPMENT.

The small Price current meter and equipment consists of five principal parts: (1) the head; (2) the tail; (3) the hanger and weights; (4) the recording or indicating device; and (5) the suspending device. In the fol-lowing descriptions the numbers in pa[illegible]

The Head.—The head consists of a [illegible] made of six conical cups (2), brazed to a horizontal frame (3). This wheel, referred to as the cups, turns in a counter clockwise direction on a vertical axis known as the cup shaft, which rests and revolves on a cone point bearing at the lower end and engages the recording mechanism at the upper end.

The cup shaft consists of two parts (4, 5), screwed together from either side of the cup frame, thus fastening the cups rigidly to the cup shaft. At the lower part of the cup shaft there is a cone bearing which receives the cone point (6) on which the cups revolve.

The cone point is screwed through a metal bushing (7) known as the cone plug, and is firmly held by a lock-nut (8). The cone plug fits into the lower arm of the yoke by a sliding connection, and is clamped in position by a set-screw. By means of a sleeve-nut (9) on the lower part of the shaft, the cups can be lifted from the cone point when the meter is not in use. This sleeve-nut has a left-handed thread, so that it will not tighten when the cups revolve.

The upper part of the cup shaft is fitted with either a worm gear or an eccentric which passes into a cylindrical chamber (10), known as the contact chamber, as it contains the mechanism for making the contact which indicates the revolutions of the cups. The construction and arrangement of both the contact chamber and the mechanism contained in it

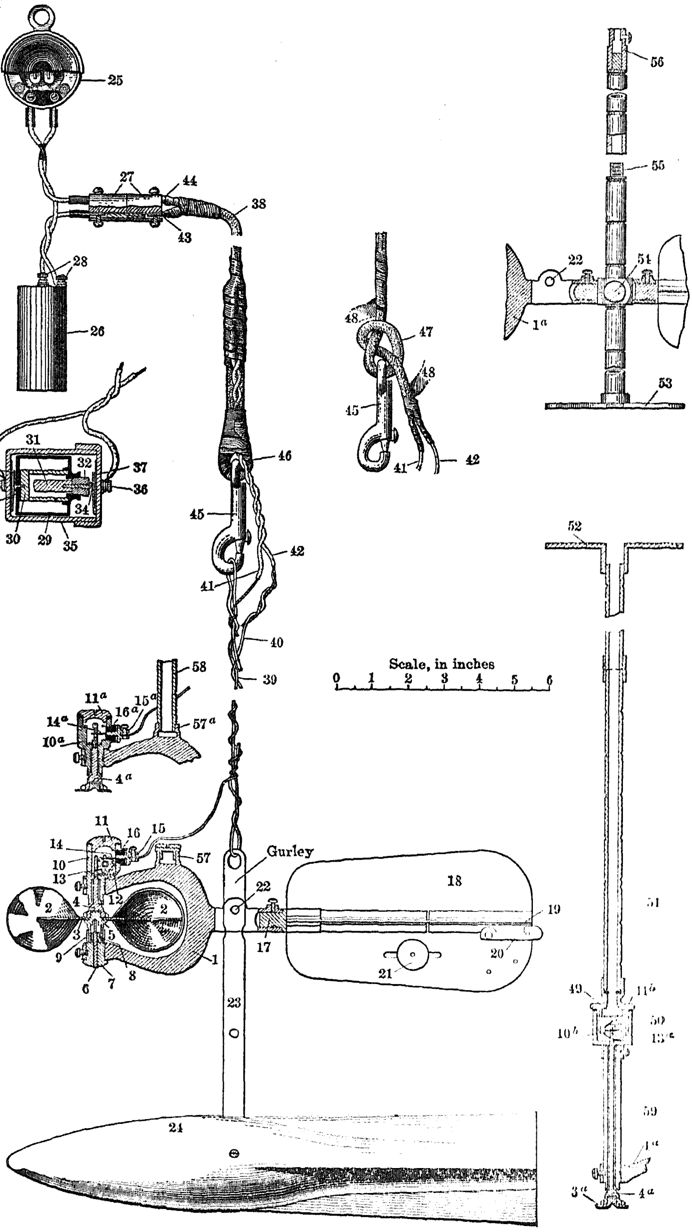


FIG. 2.—Price Current Meter and Attachments.

depend on whether the indicating device is penta-count electric, single-count electric or acoustic.

When the penta-count electric indicating device is used, the contact chamber (10) which is closed by a screw cap (11) provided with a leather gasket for keeping out the water, fits by a sliding connection into the upper end of the yoke, and is clamped into position by a set-screw. In the contact chamber there is fitted a cylindrical plug (12) which is held in position by a screw and carries a gear-wheel (13) which engages the worm gear on the upper end of the cup shaft, the gearing being arranged so that the wheel makes one revolution for every twenty revolutions of the cups. On the side of the wheel there are four platinum pins, equally spaced and set so that they will strike the contact spring (14) at each fifth revolution of the cups, thus closing the electric circuit to the indicating device, as explained later. These contact parts are known as the contact wheel, the contact pins, and the contact spring. The contact spring is of platinum, and is carried by the contact plug (15) which is screwed into the contact chamber through a hard-rubber bushing (16), thus insulating the contact spring from the meter when it is not touching one of the pins on the contact wheel. In the end of the contact plug there is a hole and a set-screw for connecting with a wire from the i

When the single-count electric indicating device is used, the contact chamber (10a) and appurtenances are the same as described for the penta-count contact chamber with the exception that the gear wheel (13) is omitted and the worm gear on the upper part of the shaft (4) is replaced by the eccentric (4a) which strikes the contact spring (14a) at each revolution, thus closing the electric circuit to the indicating device. The penta- and single-count contact chambers are interchangeable.

When the acoustic indicating device is used, the contact chamber (10b) is closed with a cap (11b) fitted with a metal drum (49), and, in place of the platinum contact spring (14) and plug (16), there is a small hammer (50) which is caused by the pins on the side of the gear-wheel (13a) to strike the drum at each fifth revolution of the cups. In order to keep the water from deadening the sound by rising into the contact chamber (10b), it is raised about four inches above the yoke (1a) by inserting the tube (59) and lengthening the upper part of the shaft (4a).

When the electric indicating device is used, the yoke is equipped with a stem which contains a slot and a screw hole (22) for attaching the meter hanger (23), and a socket into which the tail of the meter (17) is fastened. When the acoustic indicating device is used, this stem is omitted and the meter is supported on a rod (51) attached to the contact chamber.

The Tail.—The tail is used when the meter is suspended by a cable, or

on a sliding hanger rod. It provides for balancing the head, and also keeps the axis of the meter parallel to the direction of the current. It consists of a stem (17) which fits by a sliding connection into a socket in the stem of the yoke where it is clamped by a set-screw. On this stem there are two vanes (18 and 19) set at right angles. One of the vanes is rigidly attached to the stem; the other fits into it by grooves, so that it can readily be pulled out when the key (20) which holds it in place is turned. On one of the vanes there is a slot carrying a weight (21) which can be so adjusted as to balance the meter.

The Hanger and Weights.—When suspended by a cable, the meter is hung by a screw-bolt (22) on a steel stem (23) which passes through a slot in the stem of the yoke. The slot in the stem of the yoke is wide enough to allow the meter to swing freely in a vertical plane, and the bolt passes through the frame a little above the center of gravity of the meter, so that the latter will readily adjust itself to a horizontal position. In the upper end of the hanger there is a hole for attaching the suspending cable, and at intervals along the stem there are other holes by which the meter and lead weights may be hung. The weights (24) are of torpedo shape—this design offering the least resistance to the current—and are made in two sizes, weighing, respectively, 10 and 15 pounds. They are attached to the stem by a screw bolt. The manner of arrangement of the weights and meter on the stem depends on the conditions under which the measurements are to be made.

When the meter is used on a rod, the hanger, weights, and usually the tail are dispensed with.

The set-screws for clamping the various sliding connections are all of the same size and are of standard make. Beveled grooves are provided in each of these connections so that when the set-screws engage them the parts are drawn into place.

All parts of the meter are standard, and can readily be replaced in the field.

The Recording or Indicating Device.—A recording or indicating device is necessary for determining the number of revolutions of the meter wheel, and the successful use of the meter depends largely on this part of the apparatus. Various devices, operated either on the mechanical, electric, or acoustic principle, have been used for this purpose. These include the telegraph ticker, automatic recorder, electric buzzer, telephone receiver, drums, etc. Of these, however, the telephone attachment and the acoustic indicator have been found to be most satisfactory in general practice.

The telephone attachment consists of a telephone receiver (25) and small battery (26) placed in a partial circuit which terminates in a con-

necting plug (27) by means of which the apparatus can be readily connected in circuit with the meter. The magnets of the telephone receiver are wound for 10-ohm resistance so as to secure a loud click.

Either a dry-cell or a wet-cell battery may be used. The most satisfactory dry cell (26) which has been tested is the No. 409, "Ever Ready" cell, which is 1 inch in diameter and 3 inches long. This cell is equipped with two screw connecting posts (28), both at the same end.

The wet cell in common use consists of an outer casing of hard rubber (29), about $1\frac{1}{2}$ inches square, containing a carbon compartment (30) into which a zinc pole (31) having a rubber stopper (32) is inserted. The current is generated by means of a solution of bisulphate of mercury and water. Contact is made with the cell through a platinum plug (33) extending into the carbon at the bottom and through the screw (34) in the zinc pole which extends through the rubber stopper.

The cell is encased in a leather box (35), and connection is made with it through two screw connecting posts (36), each of which terminates in a separate spring plate (37) against which the poles of the battery bear.

In use, the telephone receiver is pinned to the shoulder and the battery cell is placed in the side coat pocket. The connecting plug (27) will then hang a little below the shoulder and is easily ac for attaching and detaching the meter.

In the acoustic indicator, the striking of the hammer (50) on the drum (49) in the contact chamber (10b) indicates each fifth or tenth revolution of the meter, as already explained. The sound is transmitted through the rods (51) and a rubber tube to the ear of the operator. The rubber tube and ear-piece are not necessary unless there is considerable noise.

Automatic recorders have been used to some extent, but for general work have not been found to be satisfactory, because they are likely to get out of order. They frequently require an assistant to operate them and make the outfit more cumbersome. Furthermore, a sounding device which requires the operator to count the revolutions of the meter is always safer and more satisfactory than either a mechanical or electric self-counting device or recorder, because the operator will at once detect any irregularities caused by trouble with the meter, battery, electric circuit, or other part of the equipment. A stop-watch is essential to the proper observation of time.

The Suspending Device.—The suspending device, which consists of a rod or of some form of cable, must make provision for lowering the meter and weight into the water and also for completing an electric circuit between the contact chamber of the meter and the recording device.

The rod in common use in connection with the electric recorder consists of a ½-inch tube (55) graduated to feet and tenths. For convenience in carrying, it is made in 1.0 or 1.5-foot sections fitted with screw threads.

Two methods of hanging the meter on the rod are in use. By the first the head and tail of the meter are attached to a sliding hanger (54), which can be moved up and down the rod or clamped in any position. On the bottom of the rod there is a flat foot (53) which keeps it from sinking into the bed of the stream, and at the top there is a plug (56) for connecting one of the wires from the recording device. The circuit between the meter wheel and the recording device is made by attaching one of the wires from the recording device to the plug in the top of the rod. The other wire follows down the rod and is attached to the contact plug of the meter. In the second method the rod (58) is connected by the screw socket (57) in the yoke.

The rods (51) for use with the acoustic indicator are of ½-inch tubing graduated to feet and tenths, and, for convenience in carrying, are made in 1.0 or 1.5-foot sections which screw together. The bottom rod connects with the contact chamber (49) by a screw, and is cut so that the distance from the center of the cups to the end of the rod is just 1.0 foot. On the upper end of the top rod there is a flat plate (52), in the center of which there is a hole through which the sound from the drum can be heard. The soundings are made with this end of the rod, and the plate keeps the end from sinking into the bed of the stream.

The best form of cable in use is a combination of No. 16, "old code, double-insulated, show-window cord" (38) and No. 12 or 14 galvanized wire (39) about which is wound a small insulated wire (40). The show-window cord is used for the upper part of the cable. It is large enough to be manipulated easily with bare hands, and, being made of two insulated wires, provides for making a circuit between the meter and the recording device. In its use, the two wires of which it is made must be separated at either end (41, 42, 43, 44) in order to make the attachment with the connecting plug (27) of the indicating device at the upper end and with the galvanized wire (39) and small wire (40) which lead to the meter at the lower end. A ring or snap (45), into which the galvanized wire is looped, is fastened, either by a loop (46) or a knot (47) to the lower end of the show-window cord.

In fastening the meter cable to the snap or ring with a knot (47), a strip of adhesive tape is wound around the cable two or three times, about 1 foot from the end, leaving about 6 inches of the tape at the beginning and end of the winding. The cable is then inserted through the snap or ring so that the snap bears on the adhesive tape, and a knot is tied in the

cable about the snap (45) and drawn down as tight as possible. The ends of the adhesive tape (48) are then wound around the cable, one above and the other below the knot, to keep it from sliding. The outside covering of the end of the cable can then be taken off to within 3 or 4 inches of the knot, exposing the ends of the two insulated wires (41, 42) which may then be fastened to the wires (39, 40) leading to the contact plug and to the hanger.

If the snap is held in a loop (46), a length of about 12 or 14 inches of the outside insulation is removed so that the wires can be doubled back and connected with those leading to the contact plug and hanger. The loop is first tied with string and then wound with adhesive tape, the tape being placed also around the cable where the ring bears on it.

The galvanized and small wires (39 and 40), which make up the lower end of the cable, should be long enough to reach from the surface to the bottom at the deepest point in the stream. Their use is advantageous because they offer small resistance to the moving water and thus reduce the distance that the meter is carried down stream.

The galvanized wire (39) provides both for carrying the weight of the meter and for one side of the circuit between the meter and the recording device. It is attached by ordinary loop connections to the snap in the lower end of the show-window cord and to the meter hanger (23). The circuit is made through it by the direct connection with the meter stem and by its connection at the upper end with one of the insulated wires (41) from the show-window cord.

The small wire (40), which provides for the other side of the circuit between the meter and the recording device, should be wound loosely around the galvanized wire in order to prevent annoying motion and wear, and may, if the water is swift, be held more securely if fastened with tire tape. At the upper end it is connected with one of the insulated wires (42) from the show-window cord, and at the lower end with the contact plug (15) of the meter. In order to aid in preserving the insulation between the galvanized and small wires they may be shellacked.

If the velocities or depths are not so great as to carry the meter down stream, the galvanized and small wires may be dispensed with. The snap (45) at the lower end of the show-window cord would then be attached directly to the meter stem and the circuit completed by attaching the insulated wires (41, 42) to the contact plug at one end and to the screw of the meter hanger at the other.

The meter may also be suspended by a single uninsulated galvanized wire, the circuit being completed through the water and ground (Fig. 3). In using the single wire the connection is from the water to the meter

through the contact point to the line, then to the battery and through the te'ephone to the bridge or cable, then to the ground and back to the water.

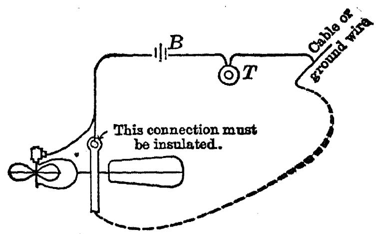


FIG. 3.—Arrangement of Circuit in Single Wire Suspension.

It makes no difference on which side of the battery the telephone is placed in the line.

When using a single wire, a clean metallic contact must be made between it and the bridge or cable from which the observations are taken. A little paint, rust, or other coating will prevent efficient work.

In measuring high velocities and deep streams, stay-lines or guy-lines are used in addition to the suspending cable to keep the meter in place.

CARE OF THE CURRENT METER.

The equipped current meter consists of:

Meter itself.

Telephone or other indicating device.

Battery.

(*d*) Connecting wires.

(*e*) Connecting plug.

(*f*) Cable for supporting the meter.

(*g*) Insulated wires for completing the circuit.

(*h*) Weights.

(*i*) Hangers.

(*j*) Hanger screws.

(*k*) Stop-watch.

(*l*) Rods for wading measurement.

(*m*) Rods or lines for sounding.

Aside from this equipment, the engineer, when on a field trip, should always be supplied with the following articles which are frequently necessary or desirable for making repairs to the station equipment and for the ordinary operation and care of the current meter.

(*a*) Small screw-driver.

(*b*) Parallel pliers with wire cutter.

(*c*) Spanner wrench for dismantling meter.

(*d*) Can of oil.

(*e*) Roll of adhesive tire tape.

(*f*) 25-foot metallic tape.

(*g*) 50-foot steel tape.
(*h*) Extra cone point.
(*i*) Extra set of screws.
(*j*) Small hatchet.
(*k*) Extra battery.
(*l*) Insulated wire.
(*m*) Assortment of nails.

For carrying the meter and equipment two types of cases are in general use. One is a box $8\frac{1}{2}$ by $6\frac{1}{2}$ by 5 inches, arranged with a shoulder strap and just large enough to carry the meter and tail when taken apart, the weights, cable, and other equipment being carried in a separate case. The other is a box 17 by 12 by 6 inches, with a lower and upper compartment, the lower being designed to carry the weights, cable, and heavier tools, and the upper to carry the meter and more delicate parts of the equipment. A partition in the upper compartment provides a space into which the head is fitted and carefully packed so as to avoid injury. This case is shaped like a small suitcase and arranged with a carrying strap.

When an additional case is needed for the equipment, the canvas hand bag, used by masons for carrying to

In taking the meter apart, remove

then loosen the set-screw to the contact chamber, and pull the chamber out by a slight twisting motion. Care must be taken to let the cups be free to turn, so that the worm gear on the upper end of the shaft can disengage from the teeth of the contact wheel. In handling the contact chamber, it is well to take off the cap, so that the gear-wheel can be seen during the operation. The cone point can then be taken out and the cups released by loosening the upper part of the shaft with a spanner wrench. This wrench is so arranged that it can be used for loosening all parts of the meter.

In putting the meter together, first attach the cups to the cup shaft. In doing this, the upper part of the shaft should be inserted through the upper hole of the yoke before it is screwed to the lower part. Care must be taken to place the cups so that they will move counter-clockwise. After the cups have been fastened to the shaft, insert the cone point and clamp it in place, and then insert the contact chamber. In replacing the contact chamber, the cups should be left free to move on the cone point and care should be taken not to allow the cogs on the worm gear to catch on the teeth of the contact wheel. Before inserting the cone plug, the cone point should be adjusted and firmly secured with a lock-nut. The adjustment should allow a slight vertical motion of the cups.

Although the current meter is substantially made and will stand con-

siderable hard usage, it needs special attention and care to insure its proper working. In this connection the following instructions should be carefully observed :

1. Be sure that the set-screws are all tightened before putting the meter in the water; otherwise one of the parts may be lost.

2. Loosen the sleeve-nut and see that the meter runs freely before beginning a measurement; and spin the meter cups occasionally during a measurement to see that they are running freely, that is, that they will continue to move for a considerable time at a slow velocity.

3. See that the weights play freely on the stem, so as to take the direction of the current and thus avoid an unnecessary drag on the line.

4. If any apparent inconsistency in the results of an observation throws doubt on its accuracy, investigate the cause at once. Grass may be wound around the cup shaft; the cups may be tilted by tension on the contact-wire; the channel may be obstructed immediately above the meter; the meter may be in a hole; or the cups may be bent so as to come in contact with the yoke.

5. After a measurement, clean and oil the bearings (in order to prevent rust) and inspect the cone point.

6. In packing the meter, turn the sleeve-nut to lift the cups from the cone point.

7. Always see that the lock-nut on the cone point is screwed firmly against the cone plug.

8. If the cone point is dulled, it can be sharpened with an oilstone.

9. In measuring low velocities, be sure that the meter is in a horizontal position. If it has a tendency to tip, the balance weight on the tail should be adjusted or the meter be held rigidly by inserting a plug in the slot against the stem.

10. Avoid taking measurements in velocities of less than 0.5 foot per second, because the accuracy of the meter diminishes as zero velocity is approached.

11. For velocities of less than 1 foot per second the bearing point should be the same as at the time of rating. As the velocity increases, the condition of the point is less important, because the friction factor decreases.

12. In taking measurements at high velocities, sufficient weight, and a stay-line, should be used to hold the meter in the vertical.

13. In very shallow streams the meter should be suspended from the lower hole on the stem, and the weight should be placed above.

14. If the cups of a small Price meter are bent, they may be easily put in shape by pressing them with a piece of wood or metal with a round, smooth end.

15. The telephone receiver is very sensitive to electric currents, and can be used to locate any break in the circuit. First try the telephone and battery together (Fig. 4) in a circuit having a make-and-break point, as at *a*. This may be done by using a knife blade or a screw-driver, making connection where the wires enter the plug. If there is no click in the telephone, then the battery or the telephone does not make a circuit. If there is a click, insert the meter in the line and test for a contact in the meter head (Fig. 5) by revolving the meter wheel. If the meter is all right, put the meter cord in the circuit and test both sides by making double connection and touching alternate sides of the line, *a* (Fig. 6).

16. When the meter is not in use, disconnect the meter line from the battery, so that it will not become exhausted.

17. When a wet cell is used, the solution may be left in it for a time, if the zinc pole and stopper are replaced by a cork.

18. Never let the bisulphate dry, however, in the cell, as it forms a hard cake and polarizes the battery.

19. Do not let any bisulphate of mercury remain loose in the meter box; if it gets into the meter bearings it will corrode them.

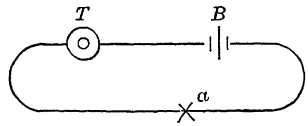


FIG. 4.

20. The zinc pole in the bisulphate cell sometimes gets pushed down so that it touches the bottom of the cell, in which case the cell is short-circuited and becomes useless. To test this, lift the plug a little way out of the battery and see if there is a flow of current.

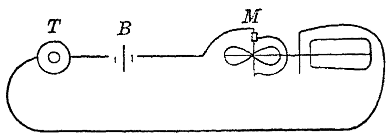


FIG. 5.

21. Keep the points clean where the battery makes contact with the metal plates.

22. The amount of current necessary to work a telephone receiver is very small, and a battery may be serviceable even though nearly exhausted.

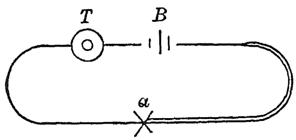


FIG. 6.

Testing Meter Circuit.

23. If care is taken, it is very improbable that the telephone receiver will get out of order.

24. Do not strike the telephone receiver, as a heavy jar will to a greater or less extent damagnetize the pole pieces, and to that extent will injure the receiver.

25. Care must be taken not to short-circuit the dry battery when the meter is not in use, as in that way the cell becomes exhausted in a short time, the energy being used in heating the cell. To avoid this, the poles may be wound with adhesive tape.

26. If a dry cell which has been long in stock fails to work well, punch two nail holes in the wax on top of the cell and put it in water over night, when it may absorb enough moisture to renew it. The holes should then be coated over by heating the wax with a match and pressing it into place, or by pouring in melted paraffin. A cell which has been exhausted by use is not benefited much by this treatment. The life of a cell depends largely on the amount of leakage in the line during use.

RATING THE CURRENT METER.

The relation between the revolutions of the meter wheel and the velocity of the water must be determined by rating each meter before it is used. Theoretically, the rating for all meters of the same make should be the same, but, as a result of slight variations in construction, and in the bearing of the wheel on the axis at different velocities, the ratings differ.

Observations for rating meter No. 315, made February 19, 1912, at Chevy Chase Lake, Maryland, by W. McC. and M. I. W. Method of suspension, Cable; meter last rated at Chevy Chase Lake, May 12, 1909; present condition good, in repair.

No. of run.	Observations for length of run.			Time in seconds.	No. of revolutions.	Revolutions per second.	Velocity per second.
	Start.	End.	Distance.				
	Feet.	*Feet.*	*Feet.*				
1	30.3	54.0	23.7	42	10	.238	.562
2	50.5	26.5	24.0	43	10	.233	.558
3	28.4	51.7	23.3	27	10	.371	.863
4	43.8	20.6	23.2	26	10	.385	.892
5	24.5	70.8	46.3	34	20	.588	1.357
6	63.9	18.1	45.8	30	20	.667	1.527
7	20.3	66.2	45.9	20	20	1.000	2.295
8	62.4	16.9	45.5	20	20	1.000	2.275
9	21.6	112.9	91.3	24	40	1.67	3.80
10	117.9	27.5	90.4	23	40	1.74	3.93
11	21.6	113.0	91.4	25	40	1.60	3.66
12	119.1	29.1	90.0	27	40	1.48	3.33
13	22.5	113.5	91.0	17.4	40	2.30	5.23
14	119.2	28.7	90.5	17.0	40	2.35	5.32
15	23.7	114.7	91.0	14.6	40	2.74	6.23
16	125.3	35.0	90.3	15.0	40	2.67	6.02

NOTE.—The runs are in pairs, the odd numbers being across the track and the even numbers in the return to the starting point.

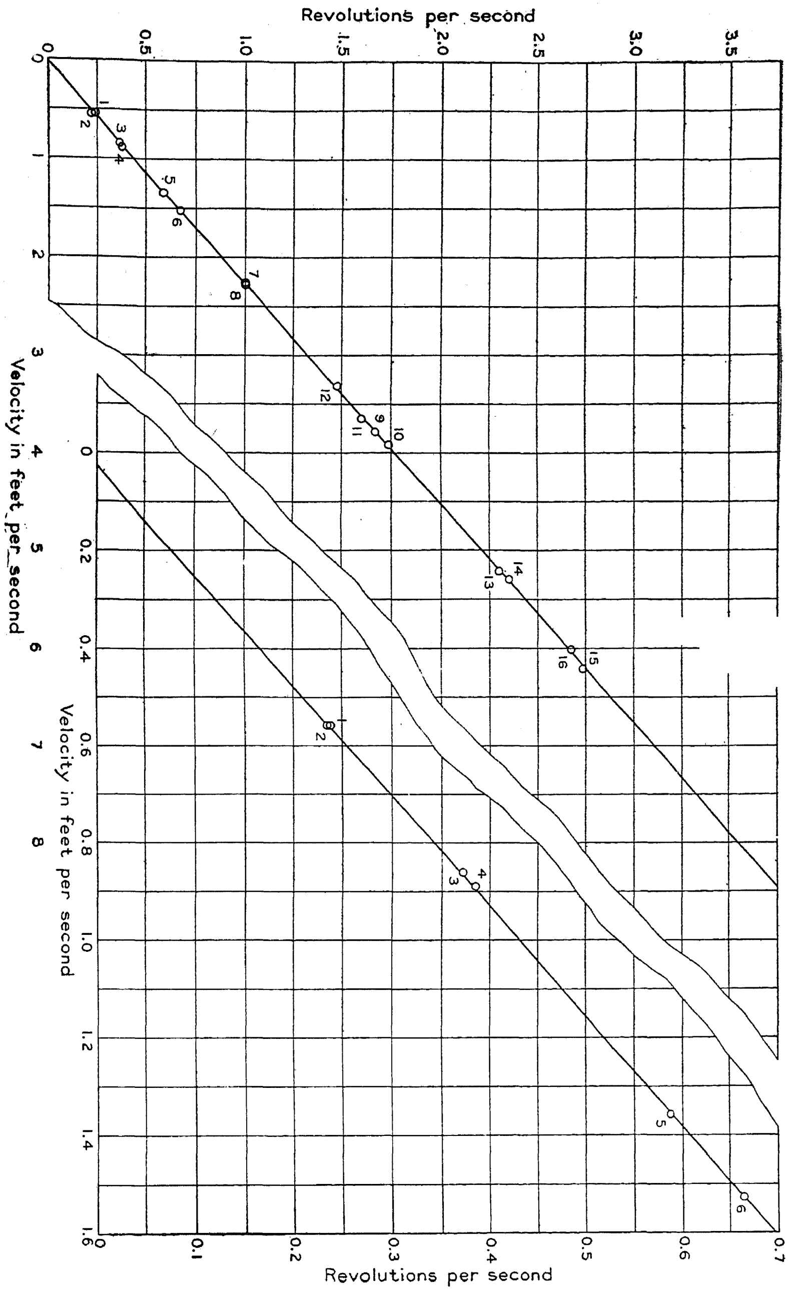


Fig. 7.—Rating Curve for Small Price Meter.

A meter is rated by conducting it through still water with uniform speed (Pl. II, A) and noting the time, the number of revolutions, and the distance. The revolutions per second and the velocity in feet per second are afterward computed from these data. Many runs are made, as shown in the preceding table, the speeds varying from the least which will cause the wheel to revolve to several feet per second. The results of these runs, when plotted (Fig. 7) with revolutions per second and velocity in feet per second as co-ordinates, locate the points which define the meter rating curve, in general a straight line from which the rating table is prepared.

In making the run for the rating the time and distance corresponding to a given number of complete revolutions are recorded automatically by electric devices which are operated by the closing of the circuit in the contact head of the meter.

Theoretically, the wheel of a differential-action meter, when carried through still water, should revolve as a wheel revolves in passing over the ground. That is, in going a given distance it should make practically the same number of revolutions, regardless of speed. The rating of a great many small Price electric meters shows this number to be from [illegible] to 44 revolutions in going 100 ft.

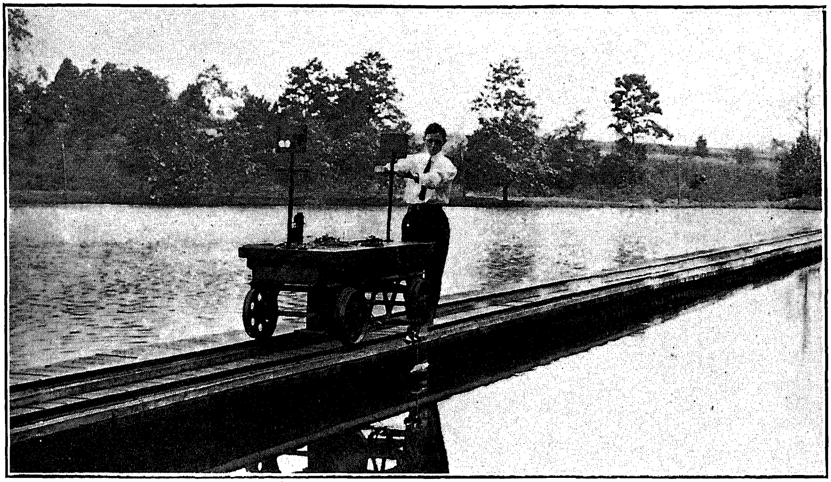
Standard current meter rating tables are usually furnished by the makers of meters and when the meters are used under the same conditions under which ratings were made, the tables will usually give results within 1 or 2 per cent of the individual rating table for the meter in question. Special ratings for individual meters can be obtained, for a nominal fee, from the United States Bureau of Standards, which maintains a fully equipped rating station at Washington, D. C.

The relative ratings of various types of current meters are shown in Fig. 8.

SOUNDING APPLIANCES.

The most common sounding appliances in general use are rods and weight and line.

Rods are limited in use to depths of less than 15 feet. If over 5 feet long, they should be round in order to be easily handled and may be made either of gas-pipe or of wood. Rods under 5 feet in length should be made of flat strips of wood 3 inches by $\frac{1}{4}$ inch with one face cut to a knife edge, against which the water will not rise in swift velocities. The graduations should be as close as the desired accuracy of soundings and so marked as to be easily read. In order to avoid sinking into the bed of the stream, the bottom of the rod should be protected by a shoe 3 inches or more in diameter.



A. UNITED STATES GEOLOGICAL SURVEY CURRENT-METER RATING STATION, CHEVY CHASE, MD.

B. TYPICAL GAGING STATION FOR BRIDGE MEASUREMENT.

Weights and lines of many forms are in use and are manipulated either directly by hand or by means of a sounding-reel in case of very deep soundings. The line should be of some material which does not shrink or stretch on wetting. For reels piano or sash-weight wire is generally used. The best form of hand line for use at bridges is a combination of the show-window cord used for supporting the meter, which can be easily grasped with the hands, for the upper part, and No. 12 or 14 galvanized wire, which offers but little resistance to the current, for the lower part.

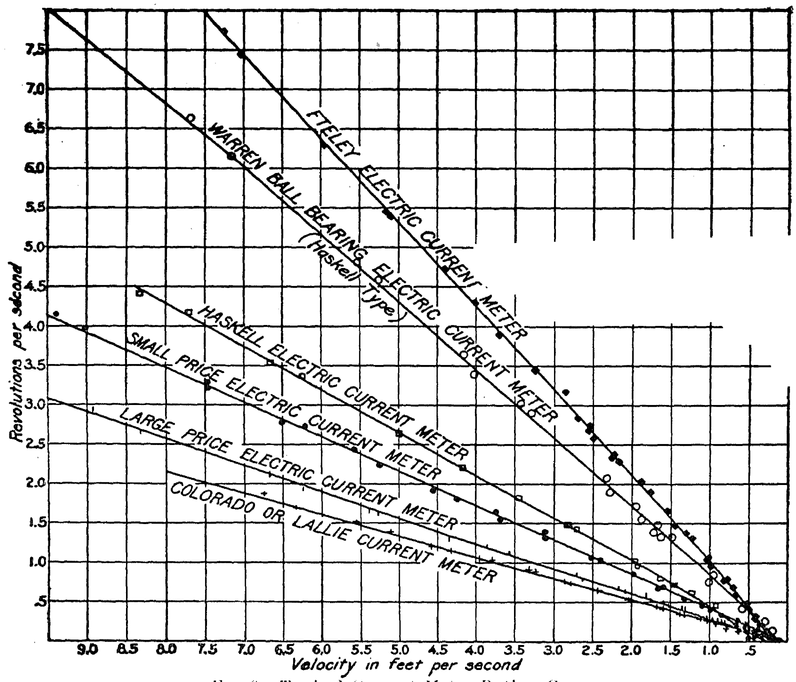


FIG. 8.—Typical Current Meter Rating Curves.

The shape of the weight should be such as to offer small resistance to the water, and the amount of weight required will depend on the depth and velocity of the current.

The line with meter and weight attached frequently is used in making soundings.

GAGES.

The gage is the instrument, graduated scale or other device, whereby the stage and changes in stage are observed or recorded. This fluctuation is measured with reference to a fixed datum which must be referred

to one or more permanent bench marks, and to which the position of the gage must maintain a constant relation. The accuracy of all records of discharge is absolutely dependent on the maintenance of this relation.

In connection with all gage height records, special care should be taken to keep a full history of each and every condition which may affect the gage records or their interpretation. These should include full notes of all matters which pertain to the gage and its installation, such as repairs and changes in datum or location, and also a history of all conditions which may affect the gage readings, such as changes in the channel or the construction of dams or other works in the vicinity.

The value of most series of gage height records increases with their length, and many long-time records have been rendered practically valueless on account of insufficient data to make possible their proper interpretation.

The many styles of gages in use all belong to two classes, non-recording and recording.

NON-RECORDING GAGES.

The various forms of non-recording gages may be grouped into (1) direct gages, consisting of fixed, graduated staffs or scale boards on which the water rises and the stage is observed directly, and (2) indirect gages, consisting of graduated scale boards located above the water surface, to which the index of the stage of the water is transferred by means of a movable chain or rod of known length operated either automatically by means of a float and counterweight or by the observer whenever a record is desired.

DIRECT GAGES.

Direct gages consist of fixed staffs which may be either vertical or inclined. If a gage of this type can be established and properly maintained, it is doubtless the most satisfactory non-recording gage that can be used. The requirements for a satisfactory gage of this class are (1) that the graduations be both clear and permanent; (2) that the gage be easily accessible to read; and (3) that it be stable. It has the advantage of certainty in datum so long as the gage is undisturbed, small first cost, and simplicity in reading, but the disadvantage of being liable to disturbance or destruction by frost action or by floating ice, logs, or drift.

Vertical staff gages.—The vertical staff is better than the inclined, when there is available, either in or over the water, an artificial or natural object having a vertical face to which the gage may be attached. Such object may be a bridge abutment or pier, a wharf, a tree, or a rock.

The best form of vertical gage consists of a base of rough 2-inch by 4-

inch or 2-inch by 6-inch plank, to which a lighter plank having the graduated face may be easily fitted and nailed, with the zero at the desired elevation. The graduated plank will be found satisfactory if made in about 5-foot sections of $\frac{7}{8}$-inch by 5-inch pine, painted white, with graduations cut as V-shaped notches painted black. This facing and graduation is cheaply made, the graduations are reasonably permanent, the sections are convenient to carry and are easily installed.

Inclined staff gages.—The inclined staff is useful where there is no existing object to which a vertical staff may be attached. It should be made of 4-inch by 4-inch timber, or larger, supported at short intervals on posts or concrete piers firmly set in the ground, and should be graduated by level after being placed in position so as to give the readings directly. Such gages are especially liable to change of datum and should be frequently checked in elevation at several points. Plate V, A, shows a hook gage in the well and an inclined gage on the bank.

INDIRECT GAGES.

Indirect gages in common use are of three types, the hook, the weight, and the float. The essential requirements for gages of this type are: (1) a constant length of the intermediate part used for transferring the index of stage to the scale board, and (2) a permanent scale board so graduated and placed that it may be easily and accurately read. They are adapted for use where a fixed staff gage would be in danger of disturbance or can not be easily read.

Hook gages.—The hook gage invented by Boyden about 1840 is the most precise instrument known for the measurement of stage and will be found of value wherever determinations of stage to a hundredth of a foot or closer are desirable. By careful adjustment such a gage can be made to read to a thousandth of a foot. The value of such accuracy of reading is, however, dependent upon the same accuracy in the determination of the other factors affecting discharge. This gage consists of a vertical inversely graduated rod, carrying a hook at the bottom. The rod slides in fixed supports provided with a vernier for reading. The hook is submerged and by means of a tangent screw is gradually raised until the point just breaks the surface of the water so as to show the pimple resulting from capillary action.

A simple form of hook gage (Fig. 9) can be arranged by using a movable staff inversely graduated to feet only, with a hook on the bottom, sliding against a fixed scale 1 foot in length carefully graduated to fractions of a foot. In reading the stage the feet are indicated by the foot-

mark on the staff which is opposite the fixed foot scale from which the tenths and hundredths are read.

Hook gages arranged with verniers are applicable for use only in connection with experimental hydraulic work or with carefully adjusted sharp-crested weirs. The simple type of hook gage has a wider range of use and will be found advantageous in conjunction with automatic gages, in canals, and other channels connected with diversion works and, during low water periods, at many gaging stations where small changes in stage correspond to large percentage changes in discharge.

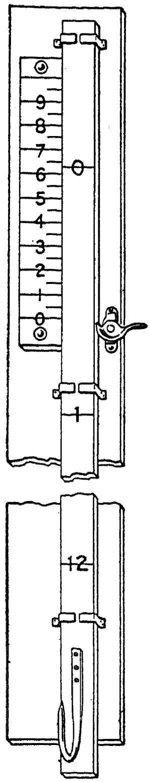


Fig. 9.—Simple form of hook gage.

Weight gages.—The simplest form of the weight gage consists of a graduated rod or tape, which the observer uses to measure vertically down to the surface of the water from a reference mark on a bridge, vertical ledge, or overhanging tree. The record of stage obtained by this means must be adjusted to read directly from the datum.

The weight gage used by the United States Geological Survey (Fig. 10) is believed to be the most practical gage of this class. It consists of a graduated scale board, 10 feet or more in length, usually either extending from or contained in a box supporting a pulley wheel, over which runs a heavy sash chain, to which is attached at one end a weight and, near the other end, a marker. This, as a whole, is fastened in a horizontal position to a bridge or other structure, so that the weight when lowered will come in contact with moving water, as the exact point of contact of the weight and water can not easily be determined by the observer above if the water is still.

Generally the scale board is graduated only for a length of 10 feet. If the range of stage is greater than that amount, provision must be made for measuring it. This is accomplished by a second and, in extreme cases, a third marker, spaced at intervals of 10 feet from the first marker.

The most satisfactory chain so far used for this form of gage is "Morton's champion metal window sash chain, No. 1 regular." Of the substitutes which have been used the best is probably some form of steel or bronze tape, which will change little if any in length but which has

been found to be liable to break, expensive to mend, and if exposed to the wind to offer considerable resistance, making it difficult to take accurate observations. Woven wire sash cord and various forms of wire are not so satisfactory as they are liable both to kink and to stretch and are not easily adjusted in length.

To read the chain gage the observer releases the chain and allows the

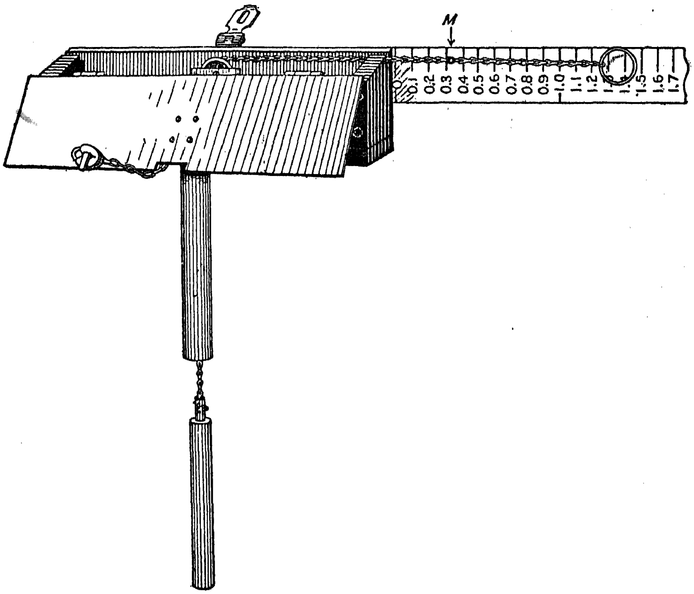


Fig. 10.—United States Geological Survey weight gage.

weight to lower until it just touches the surface of the water, in which position the stage is read on the graduated scale opposite the marker.

This gage has the advantage of stability in position, as it is above all danger from ice and drift. It has the disadvantage of possible uncertainties in the datum, on account of change in length of chain, due to wearing caused by the moving of many parts upon one another, and by changes in elevation of the structure to which it is attached. To avoid error the chain length, that is, the length from the end of the weight to the marker, must be frequently measured and adjusted to the standard length. This adjustment is made either by cutting out a link or by the adjusting device with which the chain is attached to the weight.

Float gages.—A non-recording float gage consists of a float arranged to rise and fall with the stage. The float carries either a staff or a chain passing over a pulley and kept taut by a counterweight. A marker attached to the staff or chain at a fixed distance from the float moves along a fixed graduated scale board and thus indicates the stage reading. If a staff is used on the float it may be graduated inversely and the stage observed opposite a fixed marker, or it may be arranged to read as explained for the simple form of hook gage.

This type of gage is best adapted for use in pump houses and permanent buildings erected over the water surface as, under these conditions, the float and all parts of the gage will be fully protected. When the float carries a chain, the same requirements should be observed as described for the chain on the weight gage.

ESTABLISHMENT AND MAINTENANCE OF NON-RECORDING GAGES.

In addition to the points already discussed relative to the establishment and maintenance of the various types of non-recording gages, the following conditions are generally applicable. In this connection too great emphasis can not be placed on the importance of conditions affecting the gage and its reading as the accuracy of all discharge records depends largely upon them.

Installation of gage.—The gage should be so located that it may be easily read and be without the influence of disturbing effects, such as boils, backwater, and crosscurrents. It should be graduated to read directly the elevation above the datum or zero which should be placed well below the lowest water in order to avoid negative stage readings. In order to accomplish this it is generally advisable to put the zero at the approximate elevation of the bed of the river at the lowest point in the section.

The construction and installation should be accomplished in a thorough and workmanlike manner, thus assuring the permanence of the gage and the accuracy of the results obtained by it. The scale board should be clearly graduated in accordance with the degree of accuracy expected in the observations, and each foot and tenth mark should be numbered, thereby eliminating many errors in readings. The reading of the gage should receive special consideration.

Checking gage datum.—The permanent maintenance of the datum of every gage is absolutely necessary. To accomplish this it must be referred to at least two permanent bench-marks from which it can be readily checked by means of a level.

It will be convenient if one of the bench-marks is fixed on an easily accessible part of the bridge or on an overhanging tree or rock from which the stage of the river may be directly determined by measurements made from it to the surface of the water by a staff or steel tape. Such a mark, generally known as the reference point, should be as permanent as possible and not generally any part of the gage or gage box. The other bench-marks should be placed on objects apart from the structure to which the gage is attached, out of reach of possible damage or interference and so located, if possible, that the gage can be checked with one set up of a level.

The elevation of the bench-marks should always be determined and expressed above the datum of the gage without reference to an intermediate datum.

In order that the gage heights may be readily used in flood studies and in determining slopes along the river, the datum of the gage should be, whenever possible, connected with sea level or with any city or railroad datum available.

In making the original reference and in future comparisons of the gage with its bench-marks, the level, if practicable to do so, should first be so set as to obtain directly the height of the instrument above the datum of the gage. In the case of a staff this can be accomplished by reading directly from the gage, or by setting the bottom of the level rod at some definite point on the gage. For the standard weight gage the instrument should be set below the elevation of the pulley and the gage weight lowered until its bottom is on a level with the horizontal cross-hair. The reading of the gage in this position gives directly the height of instrument.

The height of instrument should not be measured from a water surface, because the elevation of the surface of the river may vary materially within its width or within short distances up and down the stream.

In connection with the checking of indirect gages the first operation is to check and adjust, if necessary, the length of the intermediate part for transferring the index of the gage to the scale board. This having been accomplished, the datum of the gage should be compared with the bench-marks by means of a level.

If the standard chain is used, the length of the chain from the end of the weight to the marker should be measured carefully under about a 12-pound pull. In order that this measurement may be made easily the marker should be placed a few feet from the end of the chain. Nails properly spaced in the floor of the bridge will facilitate this measurement and will be serviceable in future checkings of the chain length, which should be made at each subsequent visit of the engineer to the station.

The engineer should paint or mark plainly on the inside of the cover of the gage box the length of the chain and the elevation of the reference point from which stage can be determined.

STILLING BOX.

A stilling box for eliminating wave action is desirable, and ofttimes necessary, in connection with all types of gages, especially where precise records of stage are desired, as, for example, in canals, at weirs, and at current meter stations during low water periods.

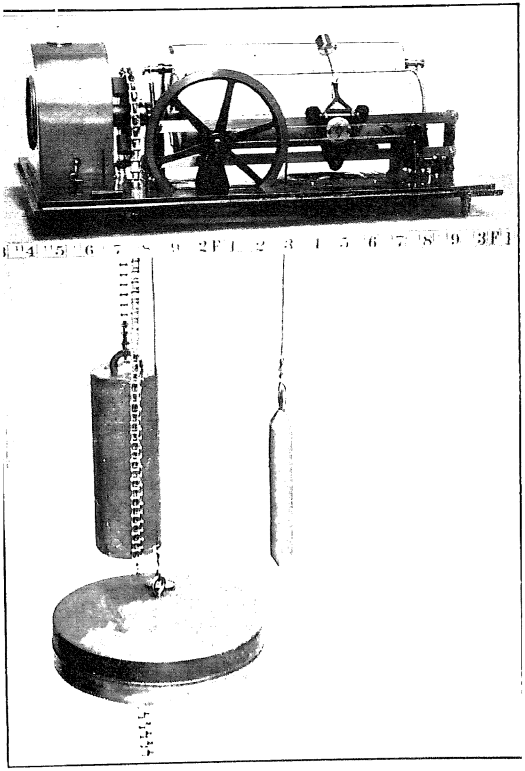
In general such a stilling box may consist of a wooden box or a metal pipe erected in the stream around the gage and extending to the bed of the stream, into which the water is admitted through small holes, or a well connected with the stream by a pipe as described for recording gages. The level of the water in the stream and in the stilling box must be frequently compared in order to eliminate errors due to the clogging of the openings to the box.

For use on staff gages, whether vertical or inclined, a special adjustable box is necessary on account of the great range of stage for which provision must be made. A wooden box that has openings through the bottom and made to slide up and down on the gage may be set at the surface of the water by the observer at the time of each observation. Frequently a tin can or pail may be utilized in a similar manner with good results. These adjustable boxes must be varied and arranged to suit particular gages.

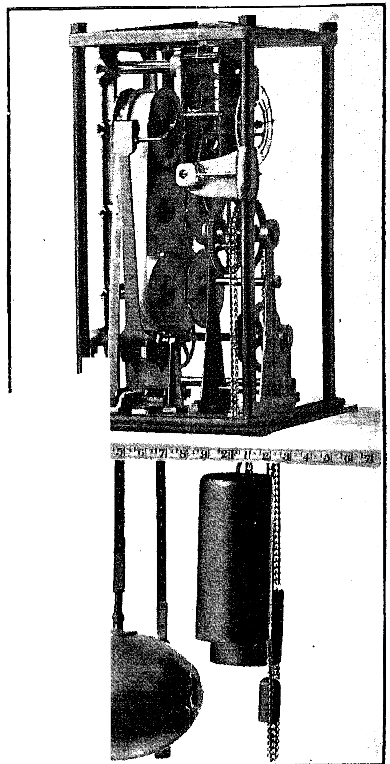
RECORDING GAGES.

Recording gages make a record of stage either continuously by a curve, the coordinates of which indicate the time and the stage, or at stated intervals of time by a printing device. The essential parts of the recording gage are: (*a*) a float which rises and falls with the surface of the water, (*b*) a device for transferring this motion of the float to the record, either directly or through a reducing mechanism, (*c*) the recording device, and (*d*) the clock.

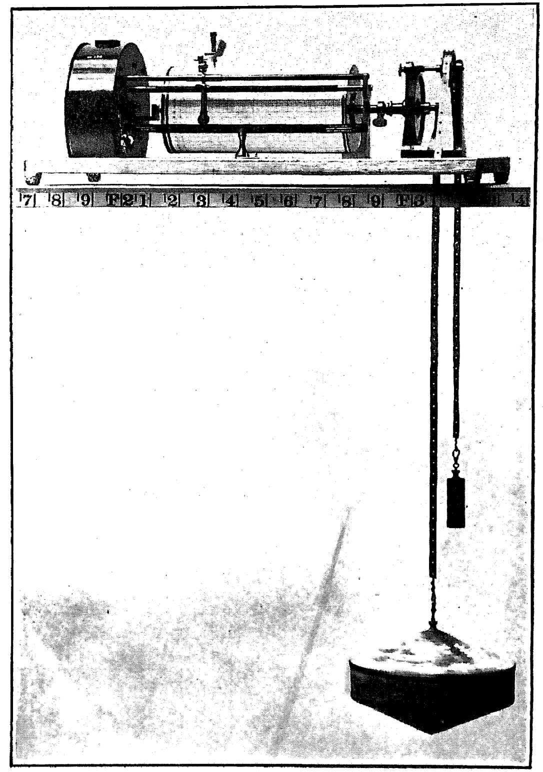
These gages should be used where the diurnal fluctuation of stage is so great and irregular that it is impossible to determine even approximately the mean daily gage height from a limited number of staff gage readings daily, as on streams artificially regulated for power or other purposes, or on those fed by melting snow or ice or subject to short and violent storms. Their use is also frequently necessary in connection with the division



A. STEVENS.



B. GURLEY.



C. FRIEZ.

RECORDING GAGES.

of water both in streams and canals, as well as on streams where daily observations of stage on a staff gage would be sufficient, but where observers are not available. They are also being used increasingly and with great benefit in the operation of power plants.

With each recording gage a staff or some other form of non-recording gage is necessary in order that the accuracy of the stage record may be easily and frequently checked.

CONTINUOUS-RECORD GAGES.

Various automatic gages have been built to record stage by means of a graph. These are generally similar in having a drum for carrying the record sheet, a movable arm for carrying the pencil or pen, a clock which propels either the pencil or the drum and determines the time ordinate, and a float with counterweight which propels either the drum or the pencil and determines the stage ordinate.

Of the many gages of this type that have been devised the Friez,[a] Gurley, and Stevens[b] (Pl. III, A and C) have been found well adapted for general use. The Friez and Gurley gages are similar in that they are operated by 8-day clocks, an signed to carry one-week records. The time ordinate is parall axis of the drum, which carries the record sheet, and the stage o is perpendicular to this axis.

The Stevens gage is operated by a weight-driven clock which can be arranged to run from 30 to 90 days by providing sufficient fall for the weight. The record sheet is furnished through a supply roll over a main drum to a receiving roll. The supply roll is arranged to carry sufficient paper for a year's record. The graph for any period of time can be removed as desired. In this gage the stage ordinate is parallel with the axis of the drum and the time ordinate perpendicular to this axis. The drum is operated by the clock, and the pencil carriage, operated by the float, is so arranged that when it reaches either limit of the gage sheet it reverses, thus recording any stage.

It is essential that continuous-record gages be arranged so that the graph will not extend beyond the limits of the record sheet. This is accomplished on the Friez and Gurley gages by having the stage ordinate pass around the drum and on the Stevens gage by the reversal of the pencil carriage.

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[a] Manufactured and sold by Julien P. Friez, Baltimore, Md.
[b] Manufactured and sold by Leupold & Voelpel, Portland, Ore.

The scales of the record sheet will depend upon the range of stage to be recorded. The usual vertical scale for large streams is 1 to 10 and for smaller streams 1 to 5. A time ordinate of an inch to a day is usually satisfactory. The possible accuracy of a continuous-record gage is determined by the reliability of the clock and the amount of lost motion in other parts of the gage, which may introduce errors in the curve.

In the operation of the continuous-record gage, visits at regular intervals are necessary in order to wind the clock and change or remove the record sheet (Fig. 11). In changing the sheet, the exact time and

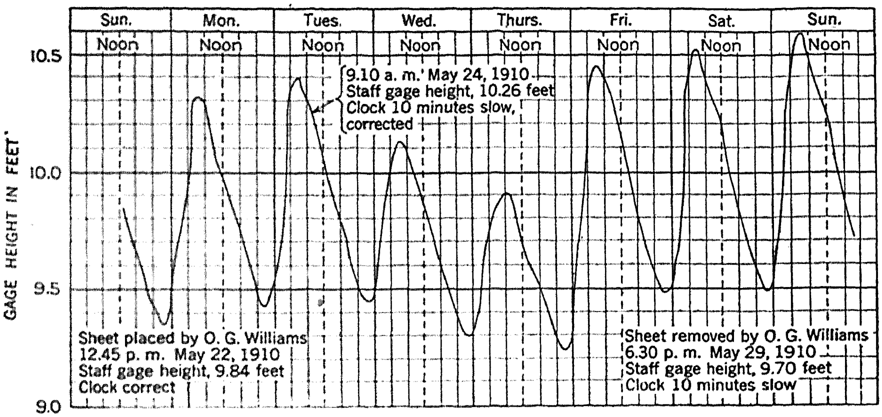


FIG. 11.—Typical Continuous-record Gage Sheet.

stage as observed on the staff gage, should be noted on the face of both the old and the new record sheets. The clock should be set, if necessary, and the amount of error in time also recorded on the old record sheet. In placing the new sheet care must be taken to start the pencil at the proper time and stage ordinates.

In addition to these regular visits, intermediate visits should be made as frequently as possible in order to insure the accuracy and continuity of the record. The exact time of such visits, together with the stage as determined by the staff gage and other pertinent notes, should be made on the face of the record sheet or in a special note book and referred to the curve by an arrow pointing to the location of the pencil point at the time of the visit. When a note book is used each entry should be numbered and dated both in the book and on the record sheet.

The continuity of a record obtained by this type of instrument does

not necessarily indicate that the record is accurate. The above precautions are therefore of importance. Full notes of all conditions which may in any way affect the record or its interpretation should be made on the record sheet or in a special note book at each and every visit of the observer.

The following sources of error are inherent in continuous-record gages of the float type:

1. Difference between elevation of water in the float well and the river.

2. Inaccurate starting of pencil on the record sheet.

3. Lag in the mechanism which prevents the recording pencil from responding promptly to changes in stage.

4. Errors in the clock.

5. Insufficient scale of both time and stage to enable accurate interpretation of the record.

6. Imperfect printing of the record sheet.

7. Expansion and contraction of the record sheet due to moisture. This can be partly eliminated by placing cubes of camphor or other absorbents in the gage box.

The mean daily gage height may be
record sheet in three ways:

1. By taking the average of readings at regular intervals of time, depending upon the variation in fluctuation of stage;

2. By means of an ordinary planimeter, in which case the area bounded by the curve and its base line is divided by the length of the base;

3. By the Fuller integrator, which gives the mean height directly by tracing the line.

In certain studies it may be desired to plot on the record sheet the curve of corresponding discharge or run-off, from which the mean daily discharge will be taken.

INTERMITTENT-RECORD GAGES.

The only successful automatic gage so far constructed which prints the stage and the time at regular intervals of time is the Gurley gage (Pl. III, B) which has been designed and built along lines suggested by the engineers of the Water Resources Branch of the United States Geological Survey. This gage is free from lost motion and the time and stage are printed (Fig. 12) to the nearest hundredth of a foot each 15 minutes. The gage is operated by a weight-driven clock and, with sufficient fall for the weight, will run 60 or even 90 days without wind-

ing. It is compact, small and comparatively simple in construction, and is adapted to work where the highest degree of accuracy is desired. The mechanism of the gage sets on an iron base 14 inches square and is covered with a tight metal cover about 21 inches high which protects it from both dust and moisture.

The recording mechanism consists of three parallel type wheels (behind the clock), on the face of which are raised figures and divisions. On the first of these wheels the periods of time from 1 to 12 hours are indicated at intervals of 15 minutes, for recording time. The height is recorded by the other two wheels, one of which carries the feet-numbers to 36 feet and the other the tenths and hundredths of a foot.

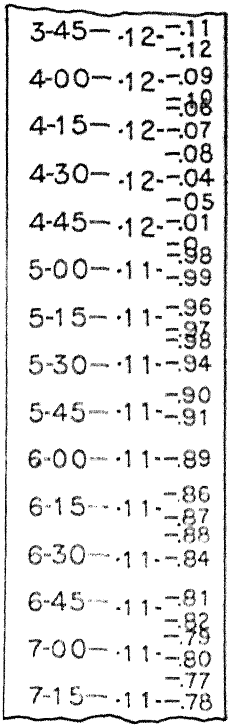


Fig. 12.—Typical Intermittent-record Gage Sheet.

The time-type wheel is controlled by a weight-driven clock, which is so constructed as to endure changes in temperature without variation in its regular operation.

The two wheels which indicate the stage of the river are actuated by a float which with its counter-weight is supported by a metal band perforated at intervals to fit over the pins in the periphery of the pulley wheel attached to the height wheel over which it runs.

The record is made by the striking of a mechanically actuated cushioned hammer, every 15 minutes, against a strip of paper which is backed with a carbon strip and passed over the face of the type wheels. The record paper and carbon paper are unwound from separate spools and taken up on two other spools, after they pass over the type wheels.

Maintenance of this gage, in addition to general inspection, requires attention in regard to the following:

1. Check the relation of water in and outside the float well.

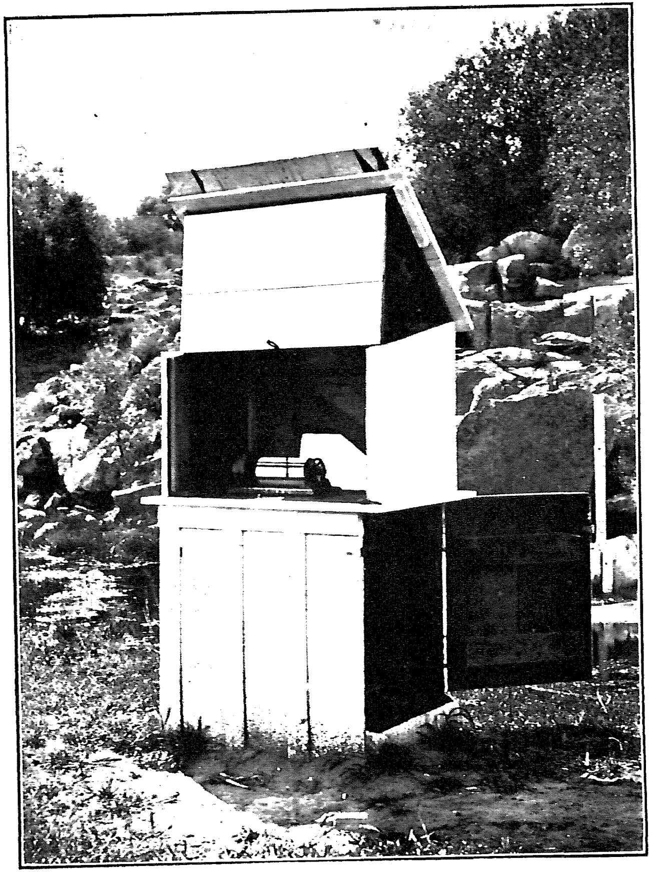
2. See that the stage-type wheel is recording correctly.

3. Check the clock.

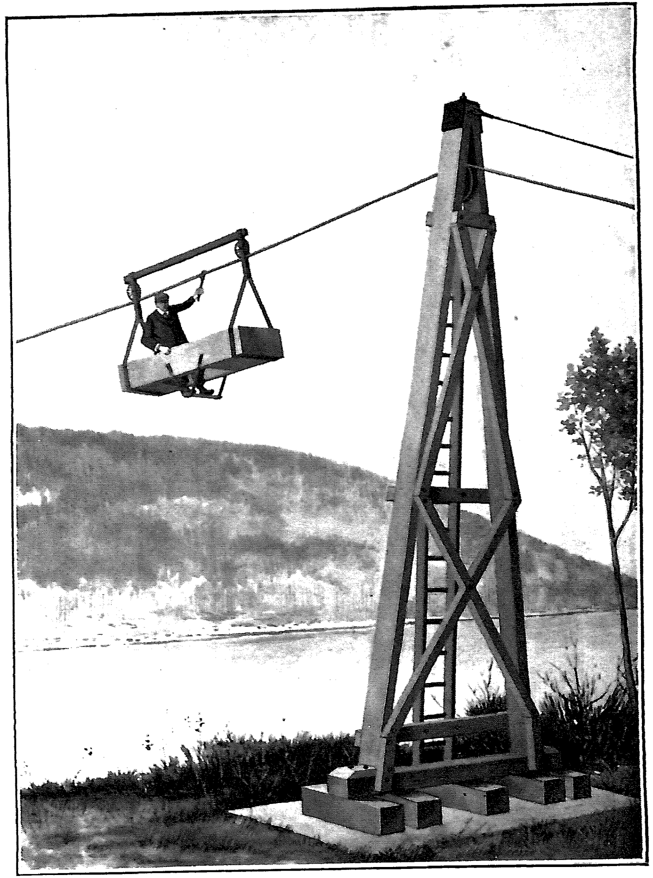
4. If desired, remove the record printed since the last visit.

5. A history of each visit, of changes, and of work done should be made in a special note book and referred to by date and number both on the record sheet and in the book.

PLATE IV.



A. BOX SHELTER FOR RECORDING GAGE.



B. CABLE TOWER AND CAR.

INSTALLATION OF RECORDING GAGES.

A large element in the satisfactory operation of any automatic gage is its proper installation, which will determine the accuracy of any recording gage record. Improper installation will deteriorate the results from the best of gages, while with an adequate installation the accuracy of the results is only limited by the construction of the gage. If the expense of an automatic gage is to be incurred, approximate results are not satisfactory and it is, therefore, essential that the installation be so thorough as to eliminate any question of the accuracy of the results. Special care and thoroughness in installation are necessary if the records are to extend over winter months and times of freshet, in order that freezing and disturbance from floating ice and débris may be eliminated.

In installing an automatic gage (Pl. V, A) it is necessary to provide a well, connected with the river, for the float; a house to shelter the gage; and staff gages with bench-marks for checking the record and maintaining its datum. Local conditions will usually determine the method and details of the installation. (Pl. IV, A.)

In the ideal installation the well and the house should be located far enough back from the river to be out of danger from floating ice or drift and to provide sufficient protection for the well and pipes to prevent freezing. The bottom of the well must be below the lowest stage and not less than 3½ feet square. It should be provided with a permanent ladder, extending to the bottom, so that the float and intake pipe can be readily inspected, and if the gage is to be maintained for a long period of time it should be lined with concrete. Otherwise a heavy plank lining can be substituted. The float pipe should be not less than 4 inches in diameter and the intake must be well below the lowest stage of the river and provided with a screen for keeping out silt, etc. It should also be provided with a check gate as it enters the well, so that the flow can be reduced to eliminate wave action. The best material for the intake pipe is spiral-welded steel with flange unions.

The shelter for the gage should have inside dimensions of at least 5 feet square and 6 feet high, in order to provide sufficient room for the observer to conveniently look after the gage. The house should have a window and the door should be closed when the cover is removed from the gage, to keep out dust. The floor should have a trap door for entering the well and a ventilating pipe should be provided both for the house and the well, in order to eliminate the dampness. The stand for the gage should be high enough to provide for its easy inspection.

The most satisfactory material for the house is concrete with metal

covering on the roof and door which will insure the gage against destruction by fire or from being otherwise disturbed. Many automatic gages in out-of-way places have been destroyed by being used as targets for rifle shooting.

Two staff gages, referred to permanent bench-marks, should be installed with each automatic gage in order to check the readings with the stage of the river. One (preferably a hook gage) should be located in the float well to determine whether the water in the well is at the same elevation as in the river, and the other should be placed in the river and of a type best suited to the locality. The river gage should be in the same cross-section of the river as the intake pipe. It may, however, be dispensed with by the use of a reference point so located that the elevation of the water surface can be easily determined from it.

When the well is properly constructed and located back from the river, there should be no danger from frost, even in temperatures as low as 30 degrees below zero. In case there is danger from freezing, it can be prevented by arranging a floating lamp in the well, or by hanging an electric light bulb near the surface of the water. Where the float is in a tube of small diameter, freezing can be prevented to some extent by pouring oil in the well.

The best type of lamp is a floating iron kettle suspended by a counterweight. In the kettle a tight cover, carrying a burner, should be soldered a few inches from the top. Such an arrangement will provide for two or three quarts of oil, which, with an ordinary lamp burner, will burn several days.

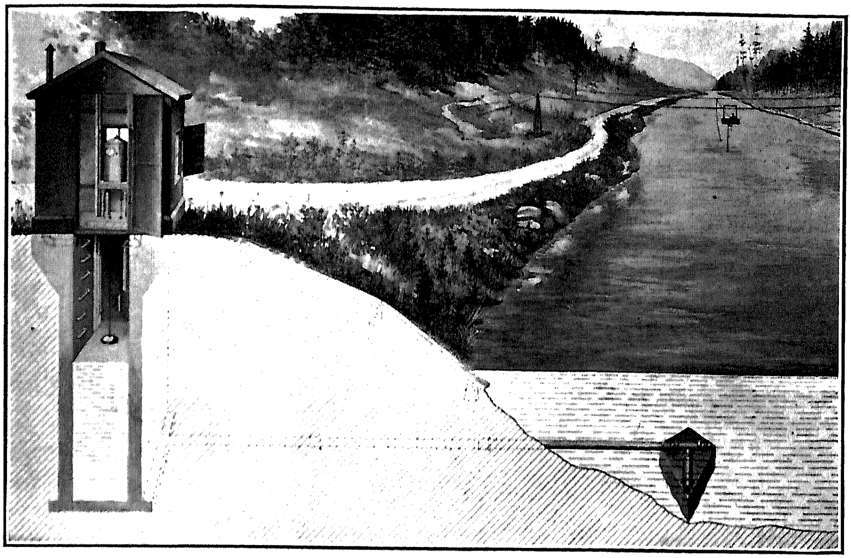
STRUCTURES FOR MAKING DISCHARGE MEASUREMENTS.

In addition to gages, as already described, regular gaging stations must be provided with—

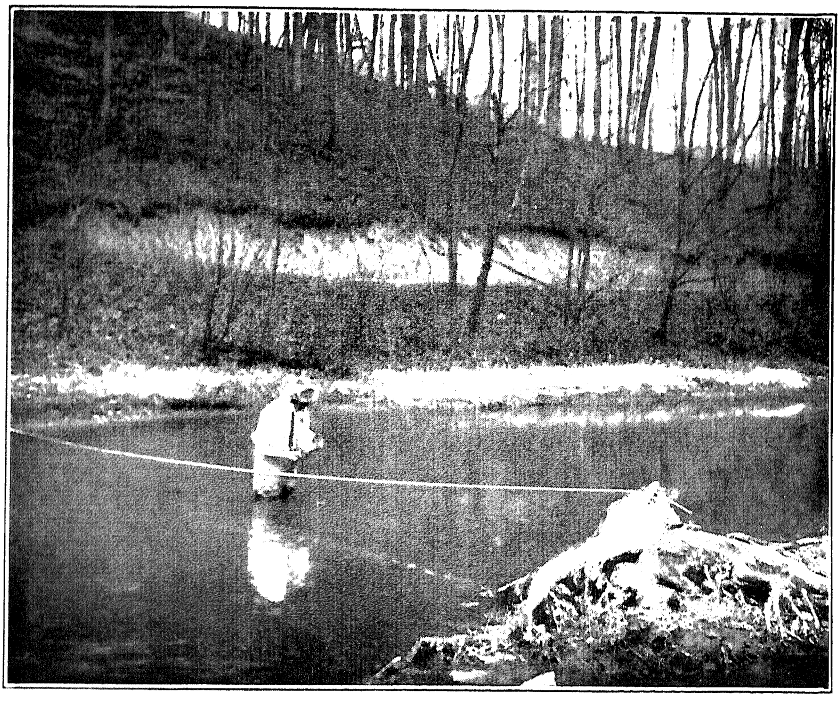
1. A structure to support the engineer while observing the velocity and depth, when the stream is too large to permit making measurements by wading.
2. A cable and stay line to hold the meter in the vertical when the soundings and velocity observations are made.
3. A graduated line for indicating the distances between the points of measurement.

STRUCTURE FROM WHICH MEASUREMENTS ARE MADE.

Discharge measurements will be made either (1) from an existing or specially constructed bridge (Pl. II, B), (2) from a cable carrying a car (Pls. IV, B, and V, A), or (3) from a boat held in position by a cable or guy line.



A. TYPICAL CABLE STATION WITH AUTOMATIC GAGE.



B. TYPICAL GAGING STATION FOR WADING MEASUREMENT.

BRIDGES.

When existing bridges are available in localities where the conditions of channel and current are suitable for the collection of a good discharge record, a gaging station may properly be located at such structure, and when so located it can generally be installed at a minimum cost. Ideal conditions for measurements are not usually found at existing bridges, and stations so located generally involve the sacrifice of accuracy to save expense. The selection of a gaging section without reference to existing structures makes possible the securing of better conditions of measurement. A material saving will be made thereby in maintenance if the station is to be continued through a considerable period of time, even though the first cost of the station is large, because fewer discharge measurements will be necessary for determining the station rating curve.

If the stream is not too large, a special cheap wooden or suspension bridge may often be constructed advantageously.

CABLES.[a]

In the absence of a bridge as a support for the engineer in making observations of velocity and depth, a cable for carrying [illegible] stretched across the stream. The equipment and appurtenances for such a cable station (Pl. IV, B) consist of the cable, supports and anchorages for sustaining it, turnbuckles for regulating the sag, and a car for carrying the observer.

The cable.—Iron or steel cable of sufficient tensile strength to sustain the car and two men, in addition to the weight of the cable itself, should be used. The stress in the cable due to a vertical load will increase as the sag decreases. Consequently the cable is least safe when the sag is a minimum. In the following table the diameter is computed for a live load of 450 pounds on the cable at the center of span and an initial tension corresponding to the sag given in the table. With an ultimate strength of 80,000 pounds per square inch the factor of safety for these dimensions is about 5. The sag given in the table is the least allowable; if it is increased, the factor of safety is increased. In making connections the cable should not be bent to a shorter radius than three diameters and the turnbuckle and connections should have a safe working strength of an amount given in the last column of the table. Galvanized cable, pulley, etc., should be used, in order to delay corrosion.

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[a] Engineering News, May 6, 1909. "The Design of Cable Stations for River Measurements," by J. C. Stevens.

Proper diameter and sag of galvanized steel cable, with live load of 450 pounds for spans of 100 to 800 feet.

Span.	Diameter.	Sag.	Stress.
Feet.	*Inches.*	*Feet.*	*Feet.*
100	$\frac{1}{2}$	4	2,938
200	$\frac{9}{16}$	6	4,167
300	$\frac{5}{8}$	8	5,061
400	$\frac{3}{4}$	10	6,300
500	$\frac{3}{4}$	12	7,813
600	$\frac{7}{8}$	12	10,125
700	1	14	12,626
800	$1\frac{1}{8}$	15	16,660

Supports.—The nature of the supports for the cable will depend on the physical characteristics of the location. It may be supported either by some natural object, as a tree or cliff, or by some form of artificial tower.

Frequently trees are properly located to serve as supports, and when so located may be cheaply and satisfactorily used. The only objection to them arises from their swaying in the wind. Protection in the form of wooden blocks must be provided for the limbs which support the cable to insure that the motion of the tree shall not speedily cause the destruction of the support. A better way, when possible, is to pass the cable through a pulley block, which, in turn, is attached to the support. Large rocks, when available at sufficient elevation above the stream bed, make excellent cable supports, as the cable can be connected directly to the anchorage.

In case artificial supports are required the form will depend somewhat on the height necessary. For low support and a short span, a single post, 10 to 14 inches in diameter, set firmly in the ground, is sufficient. When, however, heights greater than 12 or 15 feet are necessary, "shear legs" (Pl. IV, B) are generally used. In their construction two posts (8 inches by 10 inches or their equivalent in round logs) should be set in the ground 10 to 15 feet apart at the base, inclined toward each other so that they will be 2 to 5 feet apart at the top, and connected by at least three strong pieces secured to them by bolts fitted with washers and nuts or by "drift bolts" of suitable lengths; or these posts may be set so that they will cross near their ends, and should then be fastened to each other by two or more bolts with nuts. The cable may rest on the top cross-bar in the first instance or in the crotch in the second instance, but in either case should preferably be passed through a pulley block at the end having the turnbuckle. All towers should be well guyed

so they can not move toward the stream. In crossing the shear legs the cable should make equal angles with the legs on both sides.

Anchorage.—The form of anchorage will vary with different conditions. If solid rock is available, an eye-bolt split at the lower end and driven against a wedge may be set in a drill hole, which should then be completely filled with sulphur, lead, or Portland cement grout. If no solid rock is at hand, a "deadman," made of a log 8 to 12 inches in diameter, may be buried in the ground below the limits of frost and at least 4 feet deep, the length of the log and depth in the ground depending somewhat on the span of the cable.

The anchorages should be so arranged by means of long eye-bolts embedded in concrete, or auxiliary cables attached to the "deadman," that the main cable and its connections will be exposed for inspection.

The cable should be attached at each end to two independent anchorages or supports. In case posts are used for supports the cable should be attached to them by means of a short piece of cable with clips. A support which is not set in the ground should be guyed to anchors of some kind, both forward and backward, and the cable attached to it. In still other cases it is advisable to make a second independent anchorage in the ground.

Turnbuckle.—A turnbuckle for use in taking up sag, having a c of 2 to 6 feet, should be inserted in the cable on the side of the river i which the engineer approaches the station. This should have right-and-left screws and not a screw at one end and a swivel at the other.

An arrangement can easily be made whereby one man alone can tighten the cable, even if a greater length than the capacity of the turnbuckle must be taken up. This is accomplished by means of an auxiliary cable, which spans the turnbuckle and is clipped to both the main cable and the anchorage. The turnbuckle having been unscrewed and in that condition clipped to the main cable, the auxiliary cable is released and the turnbuckle drawn up. If the capacity of the turnbuckle does not remove a sufficient amount of sag, the auxiliary cable must again be clipped to the main cable and the turnbuckle released, unscrewed, and slipped along the main cable to a new position and the operation repeated.

Car.—The car should be made about 5 feet by 3 feet and about 1 foot deep and attached at each end to a pulley on the cable by means of iron or steel straps or by light cable, or by wooden standards, never by manila or cotton rope. If wooden standards are used, they should be so securely attached to the car that in case of accident they will not be wrenched loose. Plate IV, B, shows an excellent type of car. The details of the iron work for this car are shown in figure 13. The car in operation is

shown on Plate V, A. For safety and ease in propelling the car, a puller as shown on Pl. IV, B, should be provided.

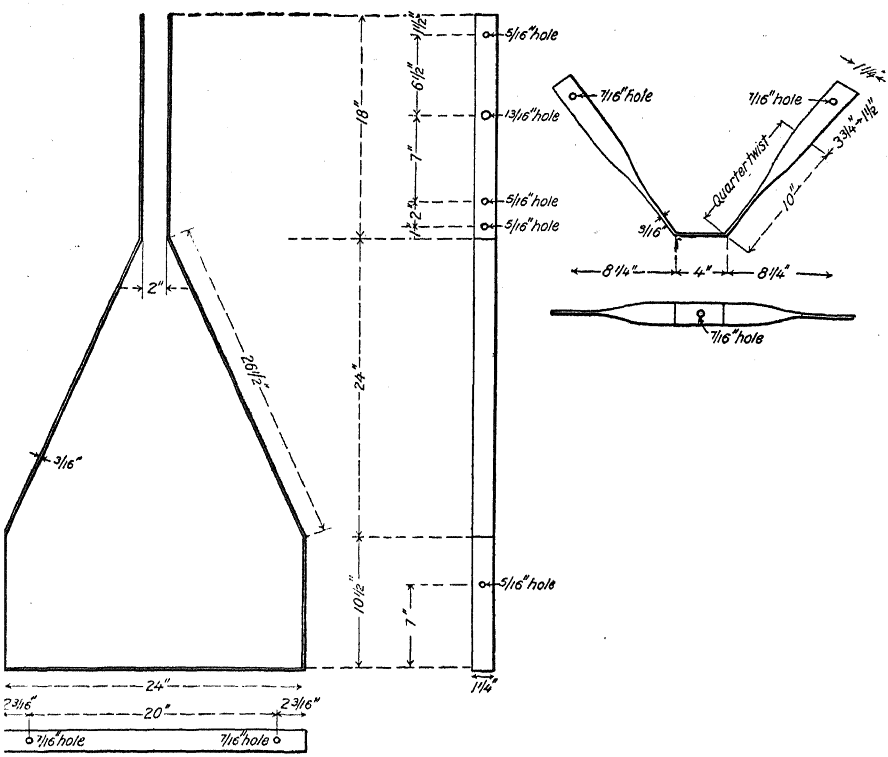


FIG. 13.—Details of Hangers for Cable Car.

BOATS.

Boat stations as ordinarily equipped are unsatisfactory on account of the difficulty in holding the boat in position, in making soundings and in operating the meter. Ferry boats operated from cables can often be advantageously used. In the measurement of large rivers, as in the work of the Corps of Engineers, United States Army, on the Niagara, St. Lawrence, and other large rivers, specially constructed catamarans[a] with special equipment for their control and operation, have been used with great success. Such equipment is expensive and is generally applicable only for special investigations on large streams.

[a] See reports of U. S. Lake Survey.

STAY LINE CABLES.

In order to hold the meter in the vertical when making measurements, all stations should be equipped with cables and stay lines (Pl. V, A). The cable need not exceed one-fourth inch in diameter and for ordinary stations a cable of No. 10 or No. 12 galvanized wire will be ample. It should be located from 30 to 100 feet above the measuring section, depending on the width and depth of the stream, and should carry a ring about 3 inches in diameter, through which a small rope is run, one end of which is connected to the upper end of the meter stem and the other end is held by the man operating the meter. In operation, the ring moves freely on the cable and the rope slides through the ring, thus enabling the observer to hold the meter in any desired position in the stream. A cable and stay line are easily installed and manipulated and are indispensable for obtaining accurate measurements when the velocities and depths are considerable.

LINES FOR INDICATING MEASURING POINTS.

In order that the measuring points at a gaging station may be easily located at the time of making measurements, and that the distance between the measuring points may be readily determined, they should be referred to a fixed initial point, and the section should be divided in regular intervals by permanent marks placed on the bridge rail or floor, in case of a bridge station; on the main cable or on a secondary tagged cable, in case of a cable station; and on a tape or tagged line stretched across the stream for measurements made from a boat or by wading. In the latter case, if it is not practicable to leave the line in place, the initial point should be so located that the line can be stretched for each discharge measurement in the same position as for previous measurements.

ARTIFICIAL CONTROL.

Streams whose beds and banks are shifting either at the gaging section or in the channel below the gage in such manner that the relation of stage to discharge is not stable or which afford no satisfactory section for making discharge measurements may require the building of artificial controls for correcting these conditions (Pl. VI, B). The practicability of constructing such controls which may be considered essential to the establishment or improvement of gaging stations on many streams, is limited by the cost of building and maintenance, both of which depend largely on the size and regimen of the stream. As generally constructed artificial controls are essentially low submerged dams but occasionally they are of the free overfall type. They vary in size and shape with the accuracy of record desired, the character of stream, the nature of bed

and banks, and the situation with respect to availability of labor and of materials for construction. In general, it will be desirable to build a control of concrete or timber well anchored to the banks and bed of the stream by means of abutments and sheet piling or other cut-off walls. A reef or bar of gravel or boulders grouted with cement may prevent change of channel and take the place of a more elaborate structure.

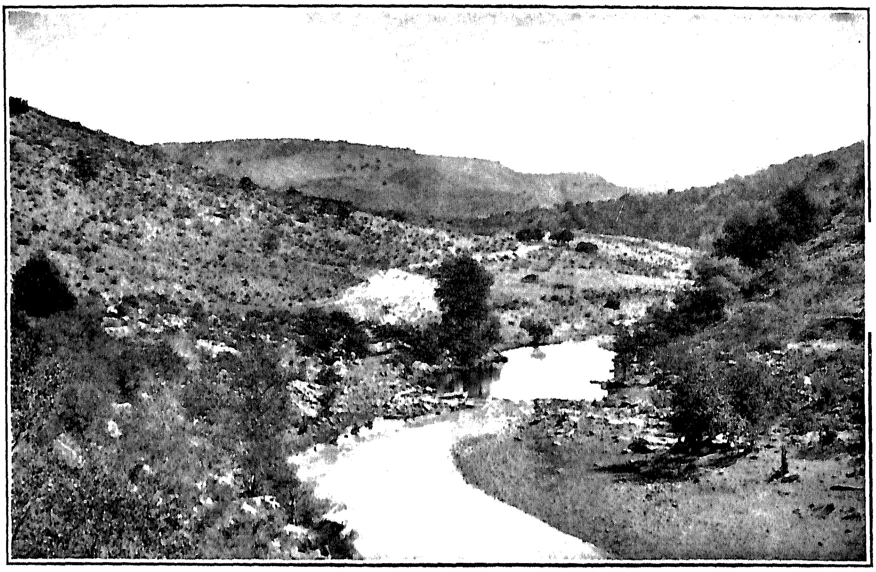
The essential features of an artificial control are (a) stability, (b) tightness, (c) sufficient height to serve as a control at all stages, (d) sufficient width in some instances to furnish a measuring section, (e) crest of such shape as to give a proper degree of sensitiveness to the station, (f) clear channel of approach, and (g) position near the gage.

Stability of the stage-discharge relation is the principal object sought in constructing an artificial control. The structure itself must, therefore, be so built that it will be stable at all stages without serious danger of failure by undermining or washing around the end and that the crest will retain its elevation and shape under the severe conditions of abrasion pertaining to streams that carry large quantities of sand, gravel, and even boulders. The conditions for stability are those pertaining to [illegible] Abrasion of crest may be reduced to a minimum by a [illegible] ıaced in the crest of a control. Such lip is most effective if placed near the downstream edge of the crest where it will serve both to give a free overfall at low stages and to hold a cover of sand and gravel on the upstream slope for its protection.

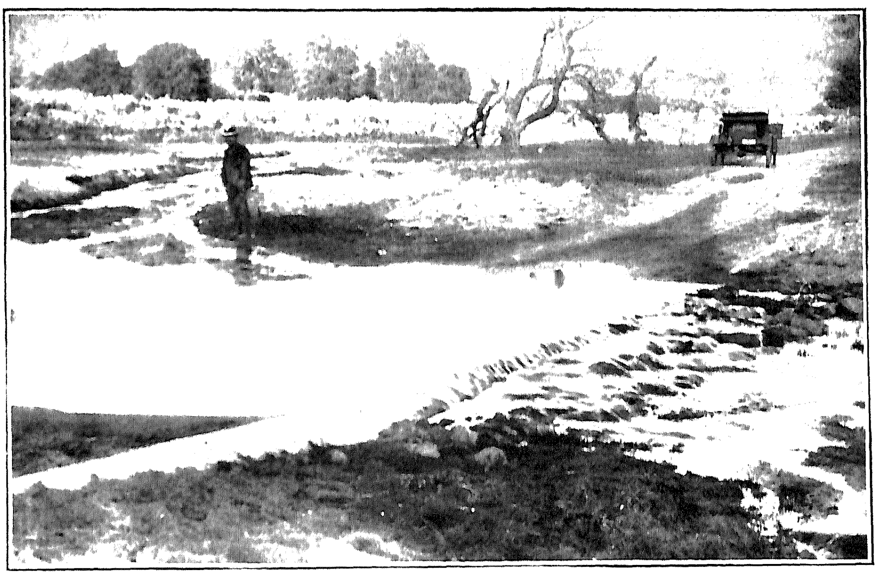
As an artificial control is constructed in order to make possible the collection of a good record of discharge by means of a record of stage, it is essential that practically all of the water shall be forced to flow where it will affect the stage. Conditions that will permit the passage of appreciable quantities of water through or around a control in strata of sand, gravel, lava or other material will not give satisfactory results.

The height necessary for an effective control will vary with the size and slope of the stream at the control and the slope and other conditions of bed and banks below. The lower the control and the less the departure from the natural conditions of channel the smaller will be the effect of running water in tearing down the structure and the cheaper will be its construction and maintenance. The height of an artificial control must be sufficient to prevent its drowning at medium and high stages by the backwater from a secondary and shifting natural control below. The smaller the slope of a stream and the more restricted and tortuous the channel below the control the greater the height needed for efficiency. A stream of rapid fall below the control offers little chance for drowning of one control by backwater from a control below. The

PLATE VI.



A. NATURAL CONTROL.



B. ARTIFICIAL CONTROL.

ratings for certain gaging stations shift only in the low-water sections. For such stations artificial controls of sufficient height to govern the stage-discharge relation throughout the range of shift will evidently accomplish the purpose as well as higher controls.

On rough mountain streams it is often difficult to find gaging sections at which reasonably accurate measurements of discharge, especially at low water, can be made. A section that is cleaned up and made reasonably smooth may fill with boulders during the next high water. On such streams it may be advisable to construct artificial controls with such width of crests, 12 inches or more, as to afford measuring sections.

The shape of the crest of an artificial control in elevation across the stream will determine the sensitiveness (p. 46) of the gaging station and therefore the accuracy of the record that can be collected. The greater the fluctuation in stage for a given change in discharge the less the refinement needed in the record of stage to obtain the desired accuracy in record of discharge. A long level spillway will create a pool that will fluctuate slowly and in which the stage must be recorded with great accuracy if reasonably accurate estimates of flow are to be made therefrom. The crest of an artificial control should therefore rise by steps or slopes from the lowest portion which may be in the center of the channel or near either bank, to
with the size of the stream and the accuracy of record desired.

The channel of approach should be clean of boulders, debris, or vegetation in order that the artificial control may be effective in maintaining a stable relation of stage to discharge at the gage. The control should be situated as near the gage as is practicable and at only sufficient distance from it to insure that the gage shall at all stages be in still water above the sharp surface slope immediately above the control.

INSTRUMENTS FOR DETERMINING CLIMATOLOGICAL DATA.

In the study of the hydrology of a given area the engineer will often need to consider the climatological conditions which affect the quantity and distribution of the water supply. In such studies it may be necessary to collect data in regard to :—(1) Precipitation in the form of rain; (2) precipitation in the form of snow; (3) evaporation ; (4) temperature ; (5) relative humidity ; (6) wind movements.

The methods and instruments of the U. S. Weather Bureau, representing the best practice for collecting climatological data, have been described in bulletins of that bureau, which may be obtained upon application.

CHAPTER III.

VELOCITY-AREA STATIONS.

Velocity-area gaging stations are divided, according to the method by which the velocities are measured, into current-meter, float, and slope stations. Current-meter stations are further divided, with respect to the facilities for making the observations, into bridge, cable, boat, and wading stations.

The data necessary for continuous records of stream-flow at velocity-area stations are, first, results of measurements of discharge, and, second, records of mean daily stage. The collection of such data requires four distinct procedures:

1. Selection of a site for the gaging stations,
2. Establishment and maintenance of the station,

ement of discharge, and

ition of stage.

SELECTION OF SITE.

REQUISITE CONDITIONS.

Conditions that determine the desirability of a site for a velocity-area station comprise, first, conditions that insure good measurements of discharge and stage, second, those that affect the computation of flow at times when measurements of discharge are not made, and, third, those that affect the cost of obtaining the records.

Conditions pertaining to measurements of flow and stage.—The measurement of discharge by current meter requires a fairly smooth bed and a measurable and uniform velocity of current. The velocity of the current should be uniformly distributed throughout the section, which should show no marked eddies, cross currents, or boils, and its mean should not be less than 0.5 foot per second at low stages. Measurements to be made by floats or by determinations of slope require also a straight stretch of channel, 200 to 1,000 feet long, through which the cross-section and velocity are reasonably uniform.

The site should be so selected that the gages for measuring stage may be economically installed and may be readily accessible for reading.

Conditions pertaining to computation of flow.—The essential conditions affecting the computation of flow are permanency of the relation of stage to discharge and the sensitiveness of this relation.

The elevation of the water surface of a stream flowing in an open channel is regulated from point to point by certain features that may conveniently be referred to as control sections. A control section may be a dam or weir, the crest of rapids or abrupt falls, a bar extending across the river, or, where the slope is uniform throughout, a long stretch of the river bed itself. The slope of the water surface above each control section is determined by the height of water at the control section. Computations of daily discharge are based on the assumption that the discharge of a stream for any given stage is unchanged so long as the character of the river at the control section remains unchanged, and that it varies with the stage according to some law. This relation between stage and discharge makes practicable the construction of a rating curve from a few measurements of discharge made at times which cover the range of stage. This rating curve is, therefore, the graphic representation of the formula for computing the discharge over or past the control section, and by its use the flow can be computed from records of stage at times when discharge measurements are not made. Any change in conditions at the control section will modify the relation between stage and discharge and make necessary the construction of a new rating curve; if, however, the conditions are permanent, measurements made in different years will define a curve that may be applied to a record of gage height extending over an indefinite time to obtain estimates of discharge.

A permanent control is, therefore, of prime importance for a gaging station maintained for the determination of daily discharge, as otherwise it is necessary to construct new rating curves after each shift, and if shifting is continuous, as in many streams in the Southwest, one or more discharge measurements a week may be required.

Attention is particularly called to the fact that permanence of flow past the gage is the essential condition, because the records of gage heights and the rating table pertain to the section at the gage and not necessarily to the section in which discharge measurements are made. This involves the requirement that the bed and banks at the control section shall be permanent. The shifting of a bar of sand or gravel in the channel below the gage may make a decided change in the relation between discharge and gage height, even though the cross-section at the point of the measurement remains unchanged. On the other hand, a permanent reef or ledge extending across the stream a short distance below the gage will control the relation between gage height and discharge, even though

the bed at the measuring section or at the gage may change. Stations established under such conditions of control give excellent records.

The relation between stage and discharge may be disturbed by various modifying conditions even when the natural control is permanent, and care should be taken in the selection of a site for a station to avoid those conditions, which may occur in the backwater from a dam as a result of intermittent diversions, or result from the choking of the river with flood water discharged from a tributary below, or from the temporary accumulation of ice, logs, or other drift.

For a given stream the shape and size of the control will govern the magnitude of change in stage resulting from a change in discharge. This relation is referred to as the sensitiveness of the station, and the station should be selected so as to give as large a change in stage as is possible for a given change in discharge. Estimates of flow at non-sensitive stations which have relatively small fluctuations in stage, are liable to large errors on account of lack of refinement in the determination of mean daily stage.

Conditions pertaining to cost of records.—Three principal factors enter into the cost of obtaining stream-flow records—the character of the records, the position and character of the gaging station, and the instruments and equipment to be used.

1. The character of the records will depend on the use for which the data are needed and the length of time that the observations are to be continued.

2. The position and character of the gaging station will depend on the characteristics of the regimen of the stream, the accessibility of the station, the availability of gage readers, and the permanency of the control, which determines the number of discharge measurements necessary for computing daily discharge.

3. The instruments and equipment for collecting the data will depend on the necessity for a recording gage, the availability of structures or equipment by means of which velocity and depth may be measured, and, in the absence of a suitably located bridge, conditions of bank favorable to anchorages and support for a cable.

RECONNAISSANCE.

A gaging station should be established only after a thorough reconnaissance has been made of the section of the river in which the station is to be placed. As the object of this reconnaissance is to find the best site to furnish the desired results, it should be made, if possible, during a low stage of the river and, if feasible, should be supplemented by

an inspection at a high stage. At low stages the bed can be carefully examined, and the minimum velocity can be determined, and a fair estimate of conditions at high stages can also be made. At medium and high stages it is generally impossible to examine the bed or to make any estimate of velocity at low stages.

Careful notes and sketches should describe the conditions at the localities examined, as the position of the station can not be finally chosen until all possible sections of the stream have been inspected. The notes should be complete in all details and should include negative as well as positive information.

In making the reconnaissance consideration should be given to the three requisite conditions that have been described, and the recorded information should include the following subdivisions of these topics:

1. Conditions pertaining to measurements of flow and stage, including (a) the type, dimensions, and location of the gage or gages; (b) the velocity and distribution of the current of water; (c) the bed—whether rough or smooth, permanent or shifting, and (d) the section of river available for determinations of slope when measurements of discharge are to be made by the slope method, estimated length, curvature, slope, obstructions, facilities for measurements of cross-sections, etc.

2. Conditions pertaining to computations of flow, including (a) the location and character of the control section; (b) the proximity of dams or tributaries above or below the section and their probable effects at the station and (c) the banks—shifting or permanent, wooded or clear, high or low, etc.

3. Conditions pertaining to cost of records, including (a) the accessibility of the site; (b) the availability of gage readers and their qualifications; (c) the estimated cost of establishment of the station; (d) the estimated annual cost of maintenance, and (e) the structures available for supporting the engineer in making measurements, or, in the absence of such structures, the span, supports, and anchorages necessary for a cable, with a statement whether or not all flood water passes under the structure or cable.

The selection must be determined largely by the facilities afforded for obtaining an accurate record of stage and for measuring precisely the area of cross-section and the velocity of the current.

In general it has been found economical, in the end, to establish a gaging station where conditions are good, even though the cost of installation may be relatively great, as the cost of operation and maintenance will probably be less and the records will be more satisfactory than at stations where conditions are poorer.

ESTABLISHMENT AND MAINTENANCE OF STATIONS.

The site for the gaging station having been selected, the routine of establishment and maintenance will depend on the equipment necessary, that is, on (1) gages, (2) structures for making measurements, and (3) controls. The character of the equipment and the manner of its installation will in large measure determine the accuracy and cost of the records. Money expended in the initial installation will generally materially reduce the cost of operation and maintenance, and thus reduce the cost of the records.

Gages.—The type of gage to be used will depend on the physical conditions at the site, the availability of a gage reader, the importance of the station, and the number of readings necessary for the proper determination of the mean daily stage. In general the gage or gages, with the necessary bench marks, will be first installed in the manner described on pages 23 to 36.

The determining factor in the use of a staff gage will be the availability gage reader, lack of which will render the use of a recording y, even though other conditions may be favorable for a g gage. The accuracy of the records of stage will depend largely on the ease with which the observations can be made; therefore the gage should be so placed as to be readily accessible, and the convenience and even the comfort of the person reading and caring for the gage should be insured by facilities provided in the vicinity of the gage.

If the gage is not located in the section in which meter measurements are made, an auxiliary gage or, preferably, a reference point should be placed in the measuring section, in order that a standard cross-section may be used for determining the area factor of the discharge.

It should be borne in mind that the rating curve of discharge applies to the cross-section of the river in which the gage is located. Therefore the location of the gage, once decided upon, should not be changed without good reason, as relocation will require the development of a new rating curve. If it is considered necessary or desirable to replace a gage, the new one if installed in the same section should be made to read from the same datum as the old one, unless there is some excellent reason to the contrary. Furthermore, secondary gages should be avoided except where necessary for determining slopes or the areas of cross sections at the measuring section. This precaution is necessary because, for a given change in discharge, the stage will vary differently at different cross-sections because of differences in condition of channel.

Structures for making measurements.—General discussion of structures

for making measurements appears on pages 36 to 40, but two important considerations, the safety of the equipment and the ease with which it can be used, are here mentioned. As the lives of the men who are to make the measurements will depend on the strength of these structures, liberal factors of safety should be employed and the materials used should be durable.

Controls.—As stated on page 45, a permanent control section is one of the fundamental requisites for a satisfactory gaging station. If the site to be used lacks a permanent natural control, an artificial control may be provided by grouting the bed or by building a low structure of wood or concrete, discussed on pages 41 to 43.

Description of station.—A complete description of the gaging station is necessary, covering the following topics:

1. The location referred to the nearest post office, railway station, dwelling, and tributary streams above and below, and if in a public-land State, to the smallest legal subdivision.
2. Date of establishment.
3. Name of person who established the station.
4. Name, post office address, and rate of compensation of observer.
5. Gage or gages and their bench marks.
6. Equipment from which measurements are made.
7. The control section.
8. Channel and other conditions which may affect measurements of discharge or stage.
9. Conditions which may affect estimates.

The description of each station should be accompanied by a general sketch showing the situation of the station with reference to topographic features, the observer's house, roads, towns, tributary streams, dams, and diversions. A detail sketch showing conditions at the gaging section may often be desirable.

THE MEASUREMENT OF DISCHARGE.

The discharge at velocity-area stations is obtained by measuring the area of the cross-section, and the velocity of the moving water. The area of the cross-section is determined by soundings. The velocity of the moving water is measured either directly—by observing the time of passage of a float over a measured course—or indirectly—by noting the revolutions of the wheel of a current meter or by measuring slope and using slope formulas. Discharge measurements are classed in accordance with these three methods of measuring velocity.

In making the measurement by means of a current meter or floats the area of the gaging section (Pl. II B) that is perpendicular to the thread of the current of the stream is divided into partial areas, for each of which the discharge is determined independently by multiplying its mean velocity by its area. The total discharge is the sum of the partial discharges. This computation of partial discharges eliminates the application of results obtained for conditions existing in one part of the channel to parts in which they do not apply.

AREA OF CROSS-SECTION.

The area of cross-section of a stream, the first factor in measuring discharge, depends on the contour of the bed, which is determined by soundings, and on the stage of the river, which is observed on the gage. The methods used in its determination will be the same regardless of the methods used for measuring the velocity. For current-meter stations the area of only the measuring section is required. For float and slope stations the average area throughout the portion of the river used for ... must be obtained.

... undings are made either by a graduated rod or by

... nding rods are limited in use to depths of less than 15 feet and are best adapted for use at wading and boat stations, where the depths and velocities are relatively small, but may occasionally be used at bridge stations where the bridge is not high above the water.

The weight and line are used in making soundings in water of greater depth than 15 feet, and from bridges or cables which are high above the water. Soundings from a bridge or cable with weight and line are most readily taken as follows: Lower the weight and line until the weight rests on the bed of the river directly underneath the measuring point. With the line taut, mark a point on it opposite a fixed point on the bridge or car; then raise the weight until it just touches the surface of the water and measure the length of the sounding line that passes the fixed point mentioned above. The depth is most readily measured by placing the end of a linen or metallic tape opposite the fixed starting point on the sounding line, grasping both the line and the tape in the hands, and drawing up the line and tape without permitting them to slip on each other until the weight rests on the surface of the water. The length of line thus drawn up, representing the depth of the water, can then be read directly from the tape. This measurement may usually be made by one person even when the depth is 10 to 12 feet. Where meter measurements are made from a bridge or cable, the meter

cable, with meter and lead attached, is generally used for sounding if the depths and velocities are small but care must be taken that the meter is not damaged.

The greatest and most common errors in measurements of discharge are caused by erroneous soundings. Errors in soundings by weight and line are due to the weight being carried downstream, so that it does not fall immediately below a point perpendicularly beneath the measuring point, or, sometimes, to the bowing of the line. Both these causes make the soundings too great. Errors in soundings with rods are due to the rod not being perpendicular, to the water rising on the rod, and to the rod sinking in the bed.

Standard cross-section.—For gaging stations on streams whose beds are permanent or nearly so, a standard cross-section should be constructed from careful soundings. This cross-section should be referred to the zero of the gage, so that the depths for any stage can be found by adding the gage height to the depth below the zero of the gage. Standard cross-sections have three uses: (1) They serve as checks on future soundings; (2) they indicate changes which may occur in the bed of the stream; and (3) they may be used in determining th
times when it is impossible to make so
or other conditions.

VELOCITY.

Velocity of flowing water, indicated by V, is generally expressed in feet per second, and depends principally upon (1) surface slope of the stream, (2) roughness of the bed, and (3) hydraulic radius.

The surface slope is the fall divided by the distance in which that fall takes place, and is represented by s. It depends on the slope of the bed, the channel conditions, and the stage. It is greater for a rising than for a falling stage.

The coefficient of roughness of the bed varies for different streams and stages of the water and is expressed by n.

The hydraulic radius or hydraulic mean depth is the area of the cross-section divided by the wetted perimeter. It is usually represented by R, and can be determined for all stages from a single complete measurement of permanent cross-section.

The mean velocity of a stream is the average rate of motion of all the filaments of water in the cross-section. It is not a directly measurable quantity, being usually found by dividing the total discharge by the area of the cross-section at a given stage. Its use is generally limited to purposes of comparison.

LAWS GOVERNING VELOCITY.

A systematic study of the flow of streams shows that mean velocity is in general a function of the stage and that the distribution of velocity through the cross-section follows well-defined laws which pertain to all streams flowing in open channels and which are in the main independent

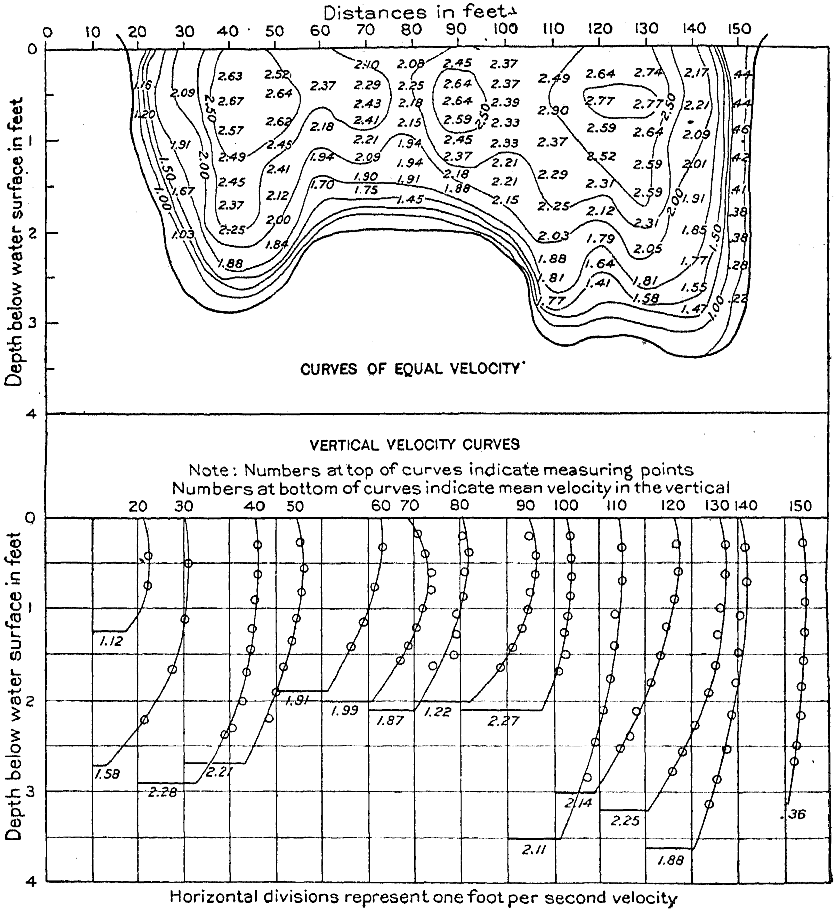


FIG. 14.—Distribution of Velocity in Open Channel, Zumbro River, Zumbro Falls, Minn.

of the stage (figs. 14, 15, 16). These laws make possible the determination of the velocity factor of the discharge measurement by comparatively few properly distributed observations of velocity. Upon them also depend the methods for determining the regimen of the stream.

These laws have been studied both mathematically and graphically

by means of vertical velocity-curves (figs. 15 and 16) which show graphically the distribution in a vertical line of the horizontal velocities of the filaments of water from the surface to the bottom of the stream.

Vertical velocity-curve.—A vertical velocity-curve is determined by a series of velocity observations taken at regular intervals in a vertical

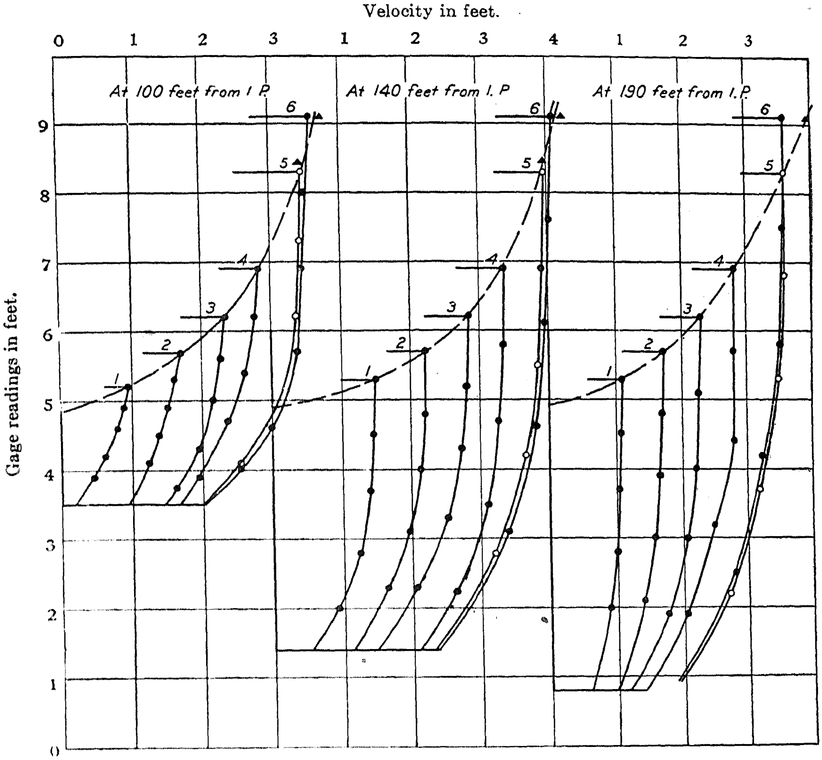


FIG. 15.—Groups of Vertical Velocity-Curves, Chenango River at Binghamton, N. Y.

from the surface to the bottom of the stream, usually from 0.5 to 1 foot apart. The results of these observations, when plotted with the velocities as abscissas and the depths as ordinates, define the curve.

Studies by Humphreys and Abbot[a] on the Mississippi, by General Ellis[b] on the Connecticut, and by the United States Geological Survey[c] on many streams under various conditions of depth, velocity, and

[a] Physics and Hydraulics of the Mississippi, 1851, p. 234.
[b] Report of the Chief of Engineers, U. S. Army, 1878, Part I, p. 259.
[c] Water Supply and Irrigation Papers Nos. 95, 109, 187, and others.

roughness of bed, show that these vertical velocity-curves have approximately the form of the parabola whose axis, coinciding with the filament of maximum velocity, is parallel with the surface and is in general situated between the surface and one-third of the depth of the water. From the maximum the velocity decreases gradually upward to the surface and downward nearly to the bottom, where it changes more rapidly on account of the friction on the bed. As the depth and velocity increase, the curve approaches a vertical line as its limiting position.

Distribution of velocity in the vertical.—If, as stated above, the velocities in a vertical line vary as the ordinates of a parabola, it may be shown mathematically that (1) a filament of water which has the same velocity as the mean of the velocities in that vertical occurs at a point between .5 and .7 of the depth measured from the surface of the stream, and (2) that the mean velocity equals the mean of the velocities occurring at .2114 and .7886 of the depth.

The demonstration[a] of the location of the filament of mean velocity is based on the theory of mean values, using the fundamental equation,

$$y = ad + d\sqrt{\tfrac{1}{3}(a^3 + b^3)} = ad + d\sqrt{\tfrac{1}{3} - a + a^2},$$

in which d equals the total depth of water; ad the depth of maximum velocity below the surface; b, the unitary complement of a; y, the depth of the thread of mean velocity.

By assigning values to a between 0 and $\frac{1}{3}d$ and substituting them with simultaneous values of b in the above equation, there results the following table, showing the depth of the mean ordinate for parabolic curves with various positions of depth to the maximum ordinates.

Depth of maximum ordinate.	*Depth of mean ordinate.*
When $a = 0$	$y = 0.58d$
$a = 0.10$	$y = 0.59d$
$a = 0.15$	$y = 0.60d$
$a = 0.20$	$y = 0.62d$
$a = 0.25$	$y = 0.63d$
$a = 0.30$	$y = 0.65d$
$a = 0.33$	$y = 0.67d$

The maximum ordinate in streams that are neither very shallow nor very deep usually lies at or above one-third depth (see table, pp. 56, 57).

[a] Engineering News, Vol. 55, p. 47, and Vol. 75, p. 889.

If it lies above one-fourth depth, the ordinate at 0.6 depth is very closely the mean ordinate. If the stream is very deep the maximum thread lies generally at a greater proportional depth, and the thread of mean velocity therefore lies at a greater depth. If a stream is shallow and has in addition a rough bed, the frictional effect on the flow is so large that the vertical velocity-curve is no longer parabolic near the bottom and the thread of mean velocity may be near mid-depth.

A study of vertical velocity-curves shows that the mean velocity in the vertical equals from 85 to 95 per cent of the surface velocity, and it also equals one-fourth the sum of the velocity near the surface plus twice the velocity at mid-depth plus the velocity near the bottom.

That these properties generally hold in nature has been proved by hundreds of vertical velocity-curves made on a large number of streams having a wide range in conditions of depth, character of beds, and magnitude of velocity. Fig. 14 shows the form of a number of typical vertical velocity-curves and the table on pages 56–57 gives a summary of results of a large number of vertical velocity-curves.

A study of these measurements, together with many others which are not available for publication, shows the general applicability in nature of the foregoing laws upon which depend t
ing discharge by floats and current mete...

METHODS OF DETERMINING MEAN VELOCITY IN A VERTICAL.

In the application of the laws of the distribution of velocity there have been developed the following methods of determining mean velocity in the vertical: (1) Vertical velocity-curves; (2) .6 depth; (3) surface; (4) .2–.8 depth; (5) three-point; (6) integration.

Measurements of velocity are therefore generally made by one of the above-mentioned methods, each of which has its special advantages and limitations. Their essential use is to determine the mean horizontal velocity in a vertical line and not features of measurement or computation that involve other factors.

As a discharge measurement contains a large number of velocity determinations the error introduced in the result by an individual erroneous measurement of velocity is generally inappreciable.

The application of the methods and the relative accuracy obtained by each, as determined by a comparative study of all available vertical velocity-curves, are discussed in the following paragraphs:

Vertical velocity-curve method.—By the vertical velocity-curve method measurements of horizontal velocity are usually made just under the

Summary of vertical velocity-curves in open channels.

Stream and locality.		Number of curves.	Range of depth—feet.	Range of velocities—feet per second.	Depth thread mean velocity in per cent of total depth.	Depth thread maximum velocity in per cent of total depth.	Coefficients for reducing to mean velocity.					
							Six-tenths depth.	Mid-depth.	Top.	Mean of top and bottom.	$\frac{.2+.8}{2}$	$\frac{T+2M+B}{4}$
Measurements since 1905.												
io	Wheeling, W. Va	14	5.3–20.8	.92–4.43	66	13	.98	......	.93	......	1.005	......
ɪongahela	Lock No. 4, Pa	7	10.9–17.5	.74–1.13	68	0	.96	......	.85	......	.990	......
Mary	Cardston, Alberta	8	2.7– 3.9	3.78–6.32	58	......	1.01	......	.88	......	1.001	......
ferson	Sappington, Mont	7	2.7– 5.7	1.78–3.45	63	......	.97	......	.84	......	.997	......
llatin	Logan, Mont	7	3.7– 5.5	.88–3.85	62	......	.98	......	.85	......	.991	......
ssouri	Cascade, Mont	22	2.1–12.0	.79–3.04	62	......	.99	......	.88	......	1.002	......
rias	Shelby, Mont	20	1.9– 4.2	.57–4.52	60	......	1.01	......	.86	......	1.004	......
llowstone	Glendive, Mont	11	4.3– 8.5	2.42–3.79	60	......	1.00	......	.89	......	1.003	......
ghorn	Fort Custer, Mont	8	3.9– 7.4	1.22–2.60	59	......	1.00	......	.86	......	.998	......
oshone	Cody, Wyo	16	4.3– 7.3	4.93–8.45	58	13	1.02	.96	.87	......	1.00	......
rth Platte	Saratoga, Wyo	8	3.9– 5.1	1.82–3.61	63	26	.98	......	.94	1.31	1.026	1.09
cos	Dayton, N. Mex	10	4.1– 8.2	1.68–2.81	58	......	1.03	......	.82	......	1.003	......
cos	Pecos, Tex	6	2.2– 6.0	1.52–3.06	63	......	.97	......	.98	......	1.004	......
een	Greenriver, Wyo	36	2.2– 6.4	1.05–4.12	58	14	1.01	......	.90	1.20	1.026	1.06
mpa	Steamboat Springs, Colo	8	4.2– 5.2	7.13–8.42	59	7	1.01	......	.83	1.10	1.005	1.02
and	Hot Sulphur Springs, Colo	6	2.8– 8.2	.89–2.87	63	23	.98	......	.85	1.30	1.01	1.03
and	Kremmling, Colo	15	9.2–27.5	1.11–4.02	65	24	.97	......	.93	1.16	1.001	1.03
nnison	Cimarron, Colo	9	4.3–15.5	3.83–8.70	64	19	.99	......	.92	1.06	1.005	1.02
nnison	Cory, Colo	5	4.9–10.4	1.63–9.59	63	17	.99	......	.92	1.09	1.007	1.01
Blackfoot	Bonner, Mont	7	4.2– 6.8	1.41–2.49	62	......	.98	......	.85	......	1.016	......
rk Fork	Missoula, Mont	5	5.4– 7.6	1.72–2.41	60	10	1.00	......	.79	......	.997	......
kima	North Yakima, Wash	7	4.9– 7.6	1.58–2.56	59	......	1.01	......	.83	1.06	.999	1.00
kima	Yakima, Wash	50	3.2–12.6	.65–8.56	59	......	1.01	......	.85	1.04	1.003	.99
kima	Kiona, Wash	20	2.0– 6.1	.25–3.24	61	......	.98	......	.84	1.05	1.00	.98
chess	Easton, Wash	5	3.4– 4.3	1.44–4.78	61	......	.99	......	.85	1.04	1.00.	.99
alum	Roslyn, Wash	20	4.7–11.7	.37–3.89	58	......	1.01	......	.84	1.09	.995	1.02
ches	North Yakima, Wash	25	2.4– 6.0	.65–7.38	58	......	1.01	......	.85	1.04	.998	.99
ke	Minidoka, Idaho	14	4.0–10.7	1.13–3.05	62	20	.99	......	.89	......	.994	......
Lost	Mackay, Idaho	7	2.4– 3.7	1.23–3.85	71	42	.95	......	.95	......	.993	......
ouse	Hooper, Wash	28	1.6– 8.0	.97–6.49	60	......	1.00	......	.84	1.06	1.004	1.00
n Day	McDonald, Oreg	15	3.6– 5.9	1.87–4.44	65	......	.98	......	.89	......	......	......
atilla	Pendleton, Oreg	13	2.2– 7.5	.47–2.73	64	......	.97	.90	.87	......	.970	......
schutes	Lava, Oreg	10	3.0– 4.1	.88–1.98	66	......	.99	.94	.92	......	.994	......
st Fork Deschutes	Lava, Oreg	27	1.7– 7.1	.58–2.65	64	......	.98	.96	.89	......	.992	......
an of 476 curves		......	......	......	62	......	.99	......	.87	......	1.001	......
ghest		......	27.5	9.59	71	......	1.03	......	.98	......	1.026	......
west		......	1.6	.25	58	......	.95	......	.79	......	.970	......

Measurements prior to 1905.												
Susquehanna	McCall Ferry, Pa	73	3.3–30.0	1.21–5.80	68	.....	.94	......	.92	1.07	....	.....
Susquehanna	McCall Ferry, Pa	68	5.0–36.0	1.40–9.70	73	.....	.97	......	.90	....	....	.....
Susquehanna	Harrisburg, Pa	20	3.2– 8.0	1.52–2.71	61	.....	1.01	......	.85	1.08	....	.....
Susquehanna	Binghamton, N. Y	36	2.5– 8.1	.80–4.86	61	.....	.99	......	......	.96	....	.....
Chenango	Binghamton, N. Y	34	1.7– 7.4	.46–3.38	62	.....	.98	......	......	1.04	....	.....
Catskill Creek	South Cairo, N. Y	47	2.1– 7.0	27–3.91	58	.....	1.02	......	.83	....	....	.....
Delaware (E. Br)	Hancock, N. Y	6	2.1– 4.8	75–4.12	59	.....	1.02	......	.82	....	....	.....
Delaware (W. Br.)	Hancock, N. Y	12	1.7– 4.2	1 67–4.45	58	.....	1.02	......	.83	....	....	.....
Esopus Creek	Kingston, N. Y	53	1.9–14.0	.65–5.04	62	.....	1.00	......	.88	....	....	.....
Esopus Creek	Olivebridge, N. Y	68	0.9– 5.2	.42–7.62	58	.....	1.01	......	.69	....	....	.....
Fishkill Creek	Glenham, N. Y	22	2.1– 5.0	.26–1.75	59	.....	1.04	......	.79	....	....	.....
Housatonic	Gaylordsville, Conn	36	2.1– 6.2	.67–3.66	58	.....	1.01	......	.83	....	....	.....
Rondout Creek	Rosendale, N. Y	21	4.3– 8.2	.35–2.55	61	.....	.99	......	.84	....	....	.....
Schoharie Creek	Prattsville, N. Y	11	1.1– 6.6	.52–3.20	58	.....	1.01	......	.80	....	....	.....
Tenmile	Dover Plains, N. Y	33	1.8– 7.3	.24–2.15	61	.....	.99	......	.88	....	....	.....
Wallkill	Newpaltz, N. Y	52	3.3–17.3	.18–4.72	64	.....	.98	......	.85	....	....	.....
Wappinger Creek	Wappinger Falls, N. Y	17	2.5– 4.7	.58–2.11	58	.....	1.03	......	.78	....	....	.....
Appomattox	Mattoax, Va	8	2.5– 4.9	.92–2.30	62	.....	1.00	......	.81	1.11	....	.....
James	Cartersville, Va	6	3.0– 4.4	1.88–2.37	60	.....	1.01	......	.88	1.08	....	.....
Roanoke	Roanoke, Va	8	1.8– 3.3	.87–1.96	61	.....	.99	......	.81	1.01	....	.....
Staunton	Randolph, Va	20	2.0– 6.0	1.35–3.11	66	.....	.96	......	.95	1.12	....	.....
Dan	Madison, N. C	42	1.3– 3.8	.73–3.00	62	.....	.99	......	.86	1.01	....	.....
Dan	South Boston, Va	13	3.5– 9.0	.90–2.41	64	.....	.98	......	.90	1.09	....	.....
Reddie	North Wilkesboro, N. C	3	1.7– 2.5	1.62–2.01	58	.....	1.02	......	.83	.97	....	.....
Yadkin	North Wilkesboro, N. C	33	2.7– 6.4	1.20–3.16	66	.....	.96	......	.91	1.12	....	.....
Yadkin	Salisbury, N. C	10	3.1–11.3	.96–3.23	59	.....	1.02	......	.81	1.06	....	.....
Catawba	Morgantown, N. C	39	1.9– 5.0	.74–2.84	61	.....	1.00	......	.82	1.04	....	.....
Catawba	Mount Holly, N. C	2	6.3– 7.0	.49– .74	58	.....	1.00	......	.88	1.08	....	.....
Catawba	Belmont, N. C	1	3.5	1.54	59	.....	1.01	......	.80	1.05	....	.....
Wateree	Camden, S. C	4	12.3–17.7	1.33–1.83	58	.....	1.01	......	.90	1.13	....	.....
Broad	Alston, S. C	4	5.0– 8.9	9–4.00	65	.....	.96	......	.89	1.22	....	.....
Saluda	Waterloo, S. C	3	3.0– 8.5	6–2.01	62	.....	.97	......	.82	1.03	....	.....
Watauga	Elizabethton, Tenn	10	2.6– 5.3	4–1.01	61	.....	.98	......	.82	1.07	....	.....
South Fork Holston	Bluff City, Tenn	13	1.9– 4.1	8–6.77	62	.....	1.02	......	.80	1.05	....	.....
Nolichucky	Greeneville, Tenn	22	1.6– 5.1	9–2.69	59	.....	1.02	......	.80	1.02	....	.....
French Broad	Oldtown, Tenn	9	4.2– 8.0	3–2.18	62	.....	.99	......	.86	1.07	....	.....
Pigeon	Newport, Tenn	4	4.4–11.6	2–1.14	61	.....	.99	......	.91	1.15	....	.....
Tuckaseegee	Bryson City, N. C	36	3.5– 8.5	0–3.39	64	.....	.97	......	.93	1.09	....	.....
Little Tennesee	Judson, N. C	11	2.7– 6.0	4–7.45	59	.....	1.03	......	.83	1.06	....	.....
Mean of 910 curves		......	....	....	61	.....	1.00	......	.85	1.07	....	.....
Highest		......	36.0	.70	73	.....	1.04	......	.95	1.22	....	.....
Lowest		......	1.1	.18	58	.....	.94	......	.78	.96	....	.....
Shallow streams with rough beds.												
St. Mary	Dam Site, Mont	24	1.8– 4.2	0–6.36	60	9	1.00	.93	.86	....	1.01	.....
Swiftcurrent Creek	Babb, Mont	16	1.6– 3.2	8–6.81	57	0	1.03	.96	.87	....	1.004	.....
South Fork Shoshone	Marquette, Wyo	20	1.8– 3.6	1–6.79	58	0	1.02	.93	.82	....	1.016	.....
Ana	Summer Lake, Oreg	6	1.8– 2.4	3–3.37	63	10	.97	.92	.89	....	1.00	.....
Tieton	North Yakima, Wash	14	1.8– 4.5	2–8.04	56	0	1.04	.96	.80	....	1.00	.98
Wallowa	Wallowa, Oreg	4	2.2– 2.9	6–4.93	58	0	1.04	.94	.78	....	1.004	.....
Kruzgamepa	Salmon Lake, Alaska	108	1.6– 3.	3–4.70	64	23	.97	.91	.86	....	1.004	.99
Small streams	Seward Peninsula, Alaska	27	1.7– 2.	5–3.47	61	14	1.00	.93	.84	....	1.004	1.00
Mean of 219 curves		......	....	....	60	7	1.01	.92	.84	....	1.005	.99
Highest		......	4.5	.04	64	23	1.04	.96	.89	....	1.016	.....
Lowest		......	1.6	.43	56	0	.97	.91	.78	....	1.00	.....

surface, at .5 foot below the surface, and at each fifth to each tenth of the depth from the surface to the bed of the stream. These measured velocities, when plotted, define for each such observation point the vertical velocity-curve from which the mean velocity in that vertical is determined.

In computing the mean velocity from vertical velocity-curve measurements, the velocity observations are plotted on cross-section paper with depths as ordinates and velocities as abscissas. A mean curve (fig. 16) is drawn through these points and extended to the surface and to the bed of the stream. The mean velocity is the mean abscissa of this curve and may be determined in three ways as follows:

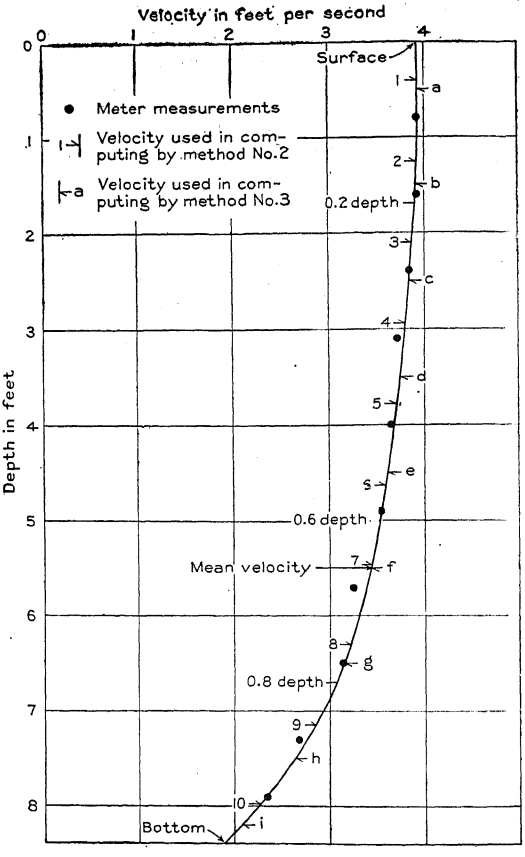


FIG. 16.—Typical Vertical Velocity-curve.

(1) Determine the area bounded by the curve and its axis with the planimeter and divide by the depth.

(2) Divide the area into any number of sections of equal depth, usually ten, and take the mean of the velocities at mid-point of each of these sections as the mean velocity.

(3) Divide the area into sections of convenient depth which will be equal except for the bottom section, which may have an odd depth. Take the mean of the middle ordinates of each section for the mean velocity. In case the bottom section is an odd depth, multiply its mean velocity by the ratio of its depth to that of the other sections, and add the product to the sum of the middle ordinates of the other sections and divide by the

number of sections, thus taking into consideration the proportionate size of the odd section.

The advantage of the third method over the second is that it enables the selection of sections corresponding in depth to a given number of divisions of the cross-section paper on which the curve is plotted. The determination of middle velocity is thus rendered much simpler than when the sections do not correspond to the divisions of the cross-section paper.

For purposes of study vertical velocity-curves are sometimes plotted with per cent of mean velocity as abscissas and corresponding depths expressed as percentages of the total depth as ordinates.

The vertical velocity-curve method is valuable as a basis for comparison of all other methods, for determining coefficients to be used in reducing values obtained by other methods to the true value, for use under new and unusual conditions of flow, and for measurements under ice. The method is not, however, in general use for making observations of velocity for routine discharge measurements, because the increased accuracy thereby obtainable is frequently overbalanced by errors arising from changes in stage of the stream during the longer time required for the measurement.

In making observations of velocity for the construction of vertical velocity-curves, velocities should also be measured at .2, .6, and .8 depth, in order that the mean velocity determined by methods in which these depths are used can be directly compared with that determined by the vertical velocity-curve method.

Vertical velocity-curves should be constructed for all stations at different stages in order to determine whether coefficients should be applied to the results obtained by the other point methods. Such application of coefficients should be made, however, only on unquestionable evidence furnished by a large number of vertical velocity-curves. The coefficient deduced from a single curve is rarely applicable to the entire cross-section of the stream.

The coefficients as determined by vertical velocity-curves for reducing the velocity by either of the other point methods to mean velocity may be plotted with stage as the other ordinate and thus determine a curve which may define the coefficient to be used at any stage.

The .6 depth method.—In practical measurement of stream discharge it is necessary to determine the horizontal velocity in a large number of verticals. Therefore, a method must be used which requires not more than three velocity observations in each vertical. If one point is used it is desirable that it be in such position that the use of a coefficient is not necessary to determine the mean velocity. The foregoing theory

shows that such a point lies approximately at .6 depth of the stream. The preceding table shows that the thread of mean velocity lies between 56 per cent and 73 per cent of the depth, with an average of 61 per cent. The error resulting from the use of .6 depth is very small, ranging from —6 per cent to +4 per cent, with a mean of 0 per cent. Therefore, in the .6 depth method it is assumed that the velocity at .6 depth is the mean velocity in the vertical and the meter is held at that point in this method.

Although this method is intended to be used without coefficients it may be found by vertical velocity-curve measurements that a coefficient is necessary in some instances to reduce the observed velocities to the mean. The method is applicable over a wide range of conditions, is easy of execution, and is reasonably accurate for normal flow in the straight reaches of all streams except very deep and very shallow ones.

The surface method.—The surface method is used in the measurement of velocities of swift streams, especially at times of freshet, when it is impracticable to sink the meter much below the surface. Therefore the observation of velocity is made at a point near the surface, but far enough below to eliminate any disturbance from wind or waves. The point of observation in this method should be from .5 foot to 1 foot below the surface, its location depending on the depth of the stream. The measured velocity must, however, be multiplied by a coefficient to reduce it to the mean. This coefficient, as shown in the preceding table, varies between 78 and 98 per cent, depending upon the depth of the stream and the magnitude of the velocity. For average streams a coefficient of about 90 per cent will generally give fairly accurate results.

The two-point method.—The two-point method is used on streams in which the location of the point of mean velocity is uncertain, or when greater accuracy is desired than can be obtained by the .6 depth method. As noted in the foregoing theory, the mean of the velocities at .2 and .8 depth gives nearly the mean velocity in the vertical. The preceding table shows that this theory holds very closely in nature. Therefore in this method the meter is held at .2 and .8 depth of each vertical.

Observations of velocity near the surface and near the bottom of the stream have in the past been used in the two-point method. Both the theory and the tables show that .2 and .8 depth should be used. This method is recommended for general stream-gaging.

Unless a measurement can be made by the vertical velocity-curve method, nothing is gained, as shown by both theory and practice, in taking velocity observations at more than two points in the vertical. In the three-point method, advocated by some, the meter is held at .2, .6 and .8 depth and the mean velocity obtained by dividing by 4 the sum of the velocities measured at .2 and .8 depth plus 2 times that at .6 depth. Such combination of the less accurate .6 depth method with the more accurate two-point method is not justified, however, as it gives results less accurate than those obtained by the two-point method.

The integration method.—The integration method is used both for obtaining the mean velocity in the vertical and also the mean velocity in the entire cross-section of the stream.

In determining the mean velocity in the vertical the meter is moved at a uniform speed from the surface of the water to the bed of the stream and return, and the revolutions and time are observed. The meter thus passes successively through all velocities in that vertical and the resulting observations determine the mean in that vertical. The method is valuable for checking other methods, but generally requires the service of at least one more man to observe time, as the engineer must be occupied with the movements of the meter. It is consequ commonly used as the point methods. The Price meter is not to observations by this method, as the vertical motion of the meter causes the wheel to revolve. The Haskell and Fteley meters, on the other hand, may be moved vertically with little or no effect on the wheel.

In determining the mean for the entire section the meter is moved with uniform speed throughout the section, usually in a zigzag path extending from surface to bottom and from side to side of the section.

CURRENT-METER MEASUREMENTS.

PROCEDURE.

In making a current-meter measurement the cross-section (Pl. II, B) is divided into partial areas, varying in width from 2 to 20 feet, depending on the size of the stream. These partial areas are bounded by perpendiculars terminating at points in the surface known as measuring points, because they indicate where the observations of depth and velocity are taken. They should be so spaced as to show any irregularities either in the cross-section or the velocity. When measurements are made at bridge or cable stations, the measuring points should be permanently marked on the bridge rail or floor, or on the cable, and used for successive measurements of discharge. When measurements

are made at boat and wading stations the points will be indicated by the graduations on a tape or tagged line, which is generally stretched at the time of each measurement.

The procedure in the measurement will vary somewhat, depending on the sounding appliance. If the meter and cord are used for sounding, observations of depth and velocity will be made at each measuring point successively across the stream. If other sounding apparatus is used, soundings will be made at all measuring points prior to taking the velocities.

In making velocity observations, one of the methods described on pages 55–61 should be used, the method chosen depending upon the conditions at the station. Care must be taken to place the center of the meter wheel at the points called for by the method. This is best accomplished by measuring the required depth on the meter line with the wheel in the surface of the water, and then lowering the meter into position. Special attention is called to the requirement, both in sounding and in placing the meter in position for observing velocity, that a tagged line should not be used for measuring depth. Such distances should be determined by means of a tape line, as indicated on page 50. In making the observations a stop-watch is desirable but not indispensable. In general, time should be noted at the click of the receiver, or at the start or finish of the buzz. The time is then observed for a given number of revolutions. The number will depend on the velocity and should be sufficient to make the time interval at least 30 seconds, as shown on the sheet of current meter notes given on pages 66 and 67. This method is preferable to observing the number of revolutions for a given time as it eliminates the error due to fractional revolutions. With a stop-watch time can be observed to half or fifth seconds.

If the velocity of the current makes other than a right angle with the measuring section the deviation from the right angle must be observed and a coefficient applied to reduce the velocity to the normal. This coefficient can usually be applied to the final completed discharge. If, however, the angle varies throughout the cross-section, it is necessary to apply appropriate coefficients to the various observed velocities. The angle can readily be determined by holding the meter just below the surface of the water and placing the notebook perpendicular to the cross-section of the stream and drawing a line parallel to the meter. This line should be divided into ten arbitrary divisions and projected upon a line normal to the gaging section. The length of this projection will be the coefficient to be used.

If the current-meter measurement of discharge is made at a regular

gaging station established for obtaining a record of discharge, a certain routine should be followed, consisting of the steps indicated below in consecutive order.

1. Check gage datum if facilities are available.
2. Set up meter, using precautions described on page 18.
3. Read the gage.
4. Make the observations necessary for the measurement of discharge by one of the methods described on preceding pages.
5. Read the gage. (If the stream is fluctuating notably the gage should be read frequently and at regular intervals during the measurement.)
6. Check the notes to make certain that all records have been made.
7. Dismantle and pack meter, using precautions described on page 18.
8. Be careful to note under "remarks" changes in stage, backwater, wind, and other conditions knowledge of which may be of future value.
9. If possible, see the gage reader and his record, and call his attention to any lack of interest or apparent discrepancies in his work.

Each of the above observations is essential to the reliability of the record at a station. On each visit of the engineer the stage of the river should also be determined by observing the distance to the surface of the water from a reference point, as a check on the gage record.

Either temporary or permanent changes in channel conditions which affect the rating of the station should be noted and recorded in as great detail as possible. Such conditions include changes in channel in the vicinity of the station, the building of dams below, the formation of jams of logs or drift, and the extent, character, and thickness of ice. Gage readers should receive written instructions in regard to reports to be made concerning such conditions; otherwise they may make no record of the time or extent of such changes.

If the engineer can reach a gaging station in time of flood, he should as a rule remain and make measurements of discharge at each foot of stage as the river rises or falls. Observation of a single freshet may thus enable him, with a minimum expenditure, to rate the station for practically all except low stages.

In addition to the general procedure described above, special precautions and methods are necessary in connection with low water and wading measurements, high-water measurements, measurements of ice-covered streams, and measurements in artificial channels, as described on pages 69 to 76.

COMPUTATIONS.

The computations of current-meter measurements are usually made to determine: (*a*) the total area of cross-section; (*b*) the mean velocity, and (*c*) the total discharge of the stream. The observed data from which these computations are made consist of (*a*) soundings at known intervals across the stream, (*b*) the velocity determinations in the vertical at each sounding point, and (*c*) the distance between the points of measurement (see table, pp. 66 and 67).

The mean velocity, area, and discharge are computed independently for each partial area included between perpendiculars drawn from successive measuring points. The total discharge is the sum of the partial discharges thus computed. The computation of the partial discharges eliminates errors which would arise from the distribution of conditions existing in one part of the cross-section to parts in which they do not apply. The mean velocity is determined by dividing the total discharge by the total area of the cross-section.

The formulas used in connection with computations of discharge may, in general, be classed as rectilinear and curvilinear—depending on the assumption that the bed of the stream and the horizontal velocity-curve are made up of straight lines or of curves between the measuring points. A comparison of the computation of discharge measurements by various formulas has been prepared by J. C. Stevens, Member Am. Soc. C. E.[a] In this discussion it is shown that the following rectilinear formula gives the most accurate results and is readily used:

$$D = l_1 \left(\frac{d_0 + d_1}{2}\right)\left(\frac{v_0 + v_1}{2}\right) + l_2\left(\frac{d_1 + d_2}{2}\right)\left(\frac{v_1 + v_2}{2}\right) + \ldots\ldots\ldots l_n\left(\frac{d_{n-1} + d_n}{2}\right)\left(\frac{v_{n-1} + v_n}{2}\right)$$

In this formula, $d_0, d_1, d_2 \ldots\ldots d_n$ and $v_0, v_1, v_2 \ldots\ldots v_n$ are the depths and velocities at the respective measuring points, $a_0, a_1, a_2 \ldots\ldots a_n$, which are spaced at the distances $l_1, l_2, l_3 \ldots\ldots l_n$ (fig. 17). The area between the perpendiculars drawn from any two successive measuring points is equal to the mean of the depths at such points multiplied by the distance between them. Similarly, the mean velocity for the area between the two perpendiculars is equal to the mean of the velocities observed at the two perpendiculars. The product of this area by its mean velocity gives the discharge for the partial area included between the two perpendiculars. The sum of these partial areas and discharges gives the total area and total discharge. The tables on

[a] Engineering News, June 25, 1908.

pages 66 and 67 show the field notes and computations for a typical current-meter measurement.

The velocities at the respective observation points, as shown in column 6, are determined from the current meter rating table, and the mean velocity in the verticals is the mean of the velocities taken at the respective measuring points. In computing the measurement, it is not usually warranted to carry the velocity computations to more than two decimal places and the partial areas and discharges to more than one decimal place.

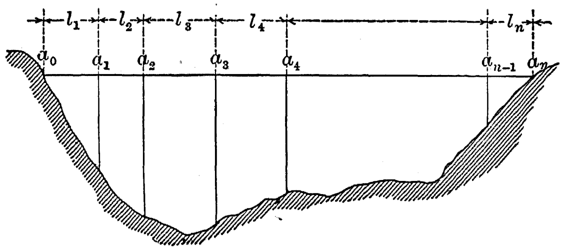


FIG. 17.—Cross-section of Stream to Illustrate Discharge Measurement Computation.

LOW-WATER AND WADING MEASUREMENTS.

At low stages of a river when the velocity is small it is advisable to find a section near by in which conditions of channel are suitable for a discharge measurement and a meter measurement may be made by wading (Pl. V, B). Low-water measurements should preferably be made by the .2–.8 depth method, except where the depth is less than 2 feet, when the .6 depth method should be used. Meters hung on rods are best adapted for use in measurements by wading.

In making the measurements a graduated line is stretched across the stream to mark the points of measurement. For this purpose a steel or metallic tape may be used. If, however, the stream is wide, an oil-silk fish line or Barbours Irish flax salmon thread, conveniently graduated, is more satisfactory as it offers less resistance to wind. When the steel tape is not used special care must be taken to check graduations to eliminate the possibility of errors due to stretching or shrinking of the line.

The engineer making velocity observations should stand below the graduated line and preferably to one side of the meter, in order not to disturb the current of the water flowing past the meter. Three-eighths-

Typical current meter notes Boise River, Dowling, Idaho. March 28, 1914.

Date March 28, 1914 No. of Meas. 35

Boise River at Dowling, State of Idaho

Width 250 Area 827 Mean Vel. 3.31 Cor. M. G. H. 4.56

Party A. B. Purton Disch. 2,740

Staff gage, checked with level and found No level available

Chain length checked with steel tape, 12-lb. pull; found ——— ft.

" " changed to ——— ft. at ——— o'clock. Correct length ——— ft.

" " corrected on basis of level to ——— ft. at ——— o'clock.

Gage	Time	Station
4.57	8:00 A.M.	33
4.56	9:00 A.M.	160
4.55	10.00 A.M.	283

ghted mean G. Ht. 4.56 ft.

orrect " " " 4.56 ft.

Meas. began 8 A.M.; ended 10 A.M.

Meter No. 898 (S.P. head)

Date rated Aug. 12, 1912

Method of meas. .2-.8 depth

No. meas. sec's 18 Coef. ---

Av. width sec. 15 Av. depth 3.3

G. Ht. change (rate per hr.) .01

— % error by — rating table.

Meas. from cable, ~~bridge, boat, wading~~. Meas. at 75 ft. ~~above~~, below gage.

~~If not~~ at regular section ~~note location~~ and conditions Good

Area from soundings (date) 3/28/14

Method of suspension Wire Stay wire No Approx. dist. to W. S. 8'

Arrangement of weights and meter; top hole —; middle hole Meter; bottom hole 20#

Gage inspected, found Solid. Cable inspected, found O.K.

Distance apart of measuring points verified with steel tape and found ———

Wind Light ~~upstr.~~, downstr., ~~across~~. Angle of current ⊥

Observer seen and book inspected Yes

Examine station locality and report any abnormal conditions which might change relation of G. Ht. to disch., e. g., change of control; ice or debris on control; back-water from; condition of station equipment Conditions apparently normal and results should be good

Sheet No. 1 of 2 sheets. ~~If insufficient space, use back of sheet.~~

Typical current meter notes Boise River, Dowling, Idaho. March 28, 1914 (continued).

Dist. from initial point	Depth	Depth of observat.	Time in seconds	Revolutions	VELOCITY At point	VELOCITY Mean in vertical	VELOCITY Mean in section	Area	Mean Depth	Width	Discharge
E. W. 3.3	0	—	—	—	—	0					
							1.35	15.0	1.25	12	20.2
45	2.5	.5	49.0	70	3.18	2.70					
		2.0	50.0	50	2.23		2.96	43.5	2.9	15	128.8
60	3.3	65	49.8	90	4.03	3.23					
		2.65	55.0	60	2.43		3.38	51.8	3.45	15	175.1
75	3.6	7	52.8	100	4.22	3.53					
		2.8	47.0	60	2.84		3.58	53.2	3.55	15	190.5
90	3.5	.7	52.6	100	4.23	3.62					
		2.9	52.0	70	3.00		3.66	53.2	3.55	15	194.7
105	3.6	.7	51.4	100	4.33	3.70					
		2.9	50.8	70	3.07		3.72	55.5	3.7	15	206.5
120	3.8	.75	54.5	100	4.08	3.74					
		3.05	52.2	80	3.41		3.64	56.2	3.75	15	204.6
135	3.7	.75	55.2	100	4.04	3.54					
		2.95	51.4	70	3.04		3.58	54.8	3.65	15	196.2
240	3.2	.65	66.0	100	3.38	2.86					
		2.55	47.6	50	2.35		2.74	45.0	3.0	15	123.3
255	2.8	.55	56.8	80	3.13	2.62					
		2.25	53.0	50	2.11		2.36	36.0	2.4	15	85.0
270	2.0	1.2	53.0	50	2.11	2.11					
							1.06	13.0	1.0	13	13.8
E. W. 283	0	—	—	—	—	0					
Totals								826.8			2,738.9

No. 2 of 2 Sheets. Comp. by A. B. P. Chk. by E. H. H.

inch iron rods, 3 or 4 feet long and having a slit in the top, are convenient for supporting the tape.

If the water is shallow or accurate determinations of small flows are desired, it may be necessary to instal sharp-crested weirs to confine the flow to a small channel in which depth and velocity will be measurable.

When convenient the soundings across the stream may be made before the observations of velocity. In making the soundings a thin, flat, graduated wooden rod should be used on which the water will not pile up, as in low-water measurements sounding errors may be relatively large. The round rods on the meter are not generally adapted to soundings. Measurements can not be made by wading if the product of depth times velocity is greater than 8.

The stage of zero flow should be determined if possible for each gaging station. This stage will be the elevation of the lowest point of the control section and should generally be determined by a level. Its great value arises in determining the position of the lower end of the station rating curve for use in estimating discharges for stages below the lowest current-meter measurement of discharge.

HIGH-WATER MEASUREMENTS.

Measurements made at high stages generally consist of observations of surface velocity only. Areas must be computed from a standard section or from soundings made at a lower stage. Under these conditions extra precautions must be taken to secure data from which a reliable estimate of the flow can be made. The great velocity and the presence of drift or of cakes of ice may render good meter measurement practically impossible. Meter observations 10 to 30 seconds long may, however, frequently be obtained when there is considerable drift, but great care must be exercised that the meter is not damaged. If a weight of more than 30 pounds is required to submerge the meter, a secondary line must be used in conjunction with the insulated cable for supporting the meter, and a stay line will often greatly assist in holding the meter in position.

When there is much drift it is generally advisable to use the float method, the drift serving as floats. Or it may be practicable to determine the slope of the river for a considerable distance and compute the discharge by means of the slope formula. If a level is not at hand marks may be made by which the slope may be determined at some future visit. When possible two of these methods may be used and a check thus obtained on the work.

When it is impossible to obtain flood measurements the discharge

may be computed as the product of the area and mean velocity, determined by extensions of the area and mean velocity curves. In such computations the area above the level of the flood plain, if any, should include only the section above the ordinary channel or channels of the stream, as the mean velocity curve usually applies only to this channel and not to the overflow channel. The discharge on the flood plain will generally be a comparatively small part of the total flow and may usually be estimated with such accuracy that the error introduced by it into the total discharge will not be great.

MEASUREMENTS OF ICE-COVERED STREAMS.[a]

The general parabolic law of the distribution of velocity in a cross section of a stream with open channel holds also for a stream under ice.

The table on pages 70 and 71, giving a summary of the results of many vertical velocity-curves, shows that there are two points in which the thread of mean velocity occurs under ice. These points are at about .1 and .71 depth below the bottom of the ice, varying between 0 and .22 for the upper and .63 and .79 for the lower. It is thus seen that they lie very nearly at .2 and .8 of the depth. As seen in the table the .2 and .8 depth method gives the mean velocity within a small percentage of error, and in making measurements under ice this method should be used.

If it is desired to make measurements having a high degree of accuracy, or for the purpose of checking results made by the two-point method, the whole measurement may be made by vertical velocity-curves (fig. 18).

When it is necessary to make observations at mid-depth a coefficient should be determined for reducing the velocity to the mean. If the river is not deep enough to get vertical velocity-curves a coefficient may be obtained at a number of stations by comparing the average of the velocity at the .2 and .8 depth with that observed at the mid-depth. See tables, pages 70 and 71, giving coefficients to reduce mid-depth velocity to mean.

In measuring an ice-covered stream the procedure is in general the same as for a stream with open channel, except that provision must be made to eliminate interference due to ice and frost. The equipment necessary for making the measurement includes the ordinary current-meter outfit, ice chisel, axe, and shovel for cutting away and removing the ice, and an ice measuring stick for determining the thickness of the ice.

The meter should be operated on a rod if the depth will permit. If,

[a] See Water-Supply Papers Nos. 187 and 337, U. S. Geol. Survey.

Summary of vertical velocity curves under ice cover.

SMOOTH ICE COVER.

River.	Station.	Number of curves.	Gage height to water surface.	Depth under ice.	Ice thickness.	Mean velocity.	Depth of threads of velocity. Upper mean.	Depth of threads of velocity. Lower mean.	Depth of threads of velocity. Maximum.	Coefficient to reduce to mean velocity. Maximum.	Coefficient to reduce to mean velocity. 0.5 depth.	Coefficient to reduce to mean velocity. $\frac{0.2+0.8 \text{ depth}}{2}$	Bed of stream.
			Feet.	*Feet.*	*Feet.*	*Ft. per sec.*							
Catskill	South Cairo, N. Y	4	2.82	2.7	0.98	0.44	0.06	0.70	0.31	0.80	0.88	0.98	Gravel.
Chemung	Chemung, N. Y	4	2.85	3.7	.51	.64	.16	.79	.46	.86	.85	1.02	Coarse gravel.
Connecticut	Orford, N. H	18	4.16	4.0	1.97	1.04	.11	.69	.37	.85	.88	1.00	Gravel and sand.
		12	±4.1	4.1	1.91	.92	.07	.63	.25	.83	.92	.99	
		6	5.6	7.3	....	1.80	.12	.73	.37	.87	.89	.99	
Esopus	Kingston, N. Y	2	5.28	5.8	.10	.94	.16	.74	.52	.87	.87	1.02	Sand and gravel.
		8	5.60	3.4	.56	.80	.03	.63	.24	.76	.85	1.01	
		12	6.6	4.9	.10	1.19	.07	.69	.29	.82	.88	.98	
		8	7.0	5.0	.50	1.26	.08	.67	.36	.82	.86	.98	
Fish	Wallagrass, Me	8	3.91	2.5	1.40	.90	.07	.70	.36	.88	.89	1.00	Gravel.
		8	5.00	3.5	1.45	1.25	.14	.77	.43	.84	.86	.98	
Genesee	Mount Morris (Jones Bridge), N.Y.	20	5.9	3.8	.54	1.62	.07	.67	.34	.80	.85	1.01	Coarse gravel.
Genesee	Rochester, N. Y	11	1.42	6.6	.45	.45	.09	.72	.44	.84	.86	.99	Do.
		19	1.55	3.8	.43	.89	[a].01	.70	.33	.82	.87	1.00	
Kennebec	North Anson, Me	4	3.40	2.7	....	2.27	....	.63	.19	.82	.92	1.00	Gravel.
		12	3.48	2.6	1.30	1.17	.04	.66	.30	.87	.91	.02	
		15	3.55	1.6	2.15	1.87	.18	.77	.45	.81	.82	1.01	
		19	4.17	2.5	1.91	1.30	.07	.69	.34	.86	.90	1.00	
		10	4.70	3.0	2.04	1.32	.10	.71	.37	.87	.89	1.00	
		9	4.77	2.9	2.34	1.33	08	.70	.32	.87	.90	.98	
		23	5.30	3.2	2.00	1.61	.12	.70	.42	.83	.86	1.01	
Mohawk	Utica, N. Y	13	[b] 16.2	10.1	.50	1.00	.21	.78	.47	.82	.84	1.04	Earth, very smooth.
Raquette	Massena Springs, N. Y	17	9.6	9.2	1.60	3.24	.17	.74	.43	.90	.88	1.00	Coarse gravel.
Rondout Creek	Rosendale, N. Y	3	5.15	6.7	.75	.55	(c)	.65	.24	.80	.89	1.00	Rock.
Wallkill	Newpaltz, N. Y	11	7.3	6.0	±1.2	.83	.11	.70	.40	.83	.87	1.01	Clay and coarse gravel.
		4	7.78	7.9	1.05	1.29	.22	.76	.46	.82	.90	1.02	
		14	8.95	7.2	1.00	1.57	.10	.70	.36	.84	.89	1.00	
		13	11.2	9.5	.85	2.35	.11	.71	.36	.85	.89	1.00	
		10	17.33	18.2	.92	3.54	.16	.79	.43	.89	.91	.99	

[a] Upper thread of mean velocity at 4 curves only.

[b] Below reference point.

[c] Upper thread of mean velocity for 1 curve only.

Summary of vertical velocity curves under ice cover—Continued.

SMOOTH ICE COVER—Continued.

River.	Station.	Number of curves.	Gage height to water surface.	Depth under ice.	Ice thickness.	Mean velocity.	Depth of threads of velocity. Upper mean.	Depth of threads of velocity. Lower mean.	Depth of threads of velocity. Maximum.	Coefficient to reduce to mean velocity. Maximum.	Coefficient to reduce to mean velocity. 0.5 depth.	Coefficient to reduce to mean velocity. $\frac{0.2+0.8 \text{ depth}}{2}$	Bed of stream.
			Feet.	*Feet.*	*Feet.*	*Ft. per sec.*							
est Canada Creek	Twin Rock Bridge, N. Y.	9	3.8	2.5	1.50	1.82	0.04	0.66	0.34	0.86	0.91	1.02	Coarse gravel.
inooski	Richmond, Vt.	26	5.5	1.6	2.10	2.12	.12	.72	.40	.80	.83	1.02	Gravel.
Mean of 352 curves							.10	.71	.37	.839	.878	1.002	
Highest							.22	.79	.52	.90	.92	1.04	
Lowest							.00	.63	.19	.76	.82	.98	

ROUGH ICE COVER.

River.	Station.	Number of curves.	Gage height to water surface.	Depth under ice.	Ice thickness.	Mean velocity.	Upper mean.	Lower mean.	Maximum.	Maximum.	0.5 depth.	$\frac{0.2+0.8 \text{ depth}}{2}$	Bed of stream.
onnecticut	Orford, N. H.	7	5.59	4.6	1.71	1.08	0.08	0.70	0.37	0.86	0.88	1.00	Gravel and sand.
		7	6.00	4.9	1.69	1.11	.09	.70	.39	.84	.88	1.00	
		21	6.70	5.8	1.66	1.23	.14	.74	.42	.86	.88	1.00	
es Moines	Keosauqua, Iowa	5	2.45	4.8	.46	1.58	.13	.73	.40	.86	.88	1.00	Gravel.
aumee	Sherwood, Ohio	11	3.34	2.5	.58	.78	.11	.68	.34	.92	.95	1.01	Do.
Mean of 51 curves							.11	.71	.38	.868	.894	1.002	
Highest							.14	.74	.42	.92	.95	1.01	
Lowest							.08	.68	.34	.84	.88	1.00	

VERY ROUGH ICE COVER, BROKEN AND TILTED.

River.	Station.	Number of curves.	Gage height to water surface.	Depth under ice.	Ice thickness.	Mean velocity.	Upper mean.	Lower mean.	Maximum.	Maximum.	0.5 depth.	$\frac{0.2+0.8 \text{ depth}}{2}$	Bed of stream.
ondout Creek	Rosendale, N. Y.	4	± 7.0	5.3	0.45	0.74	0.27	+[a]0.88	0.56	0.80	0.85	1.04	Rock.
allkill	Newpaltz, N. Y.	4	13.7	14.6	1.40	2.98	.27	.88	.56	.84	.85	1.60	Clay and coarse gravel.
inooski	Richmond, Vt.	5	5.62	6.2	±2.5	1.99	.22	.81	.52	.74	.76	1.02	Sand and gravel.
Mean of 13 curves				8.7	1.45	1.90	.25	.86	.55	.79	.82	1.02	

[a] No lower mean thread of velocity for 1 curve.

however, a cable is used, it should be tagged at foot intervals, beginning at the center of the meter for convenience in sounding and placing the meter at the proper depth.

The most satisfactory ice chisel yet found is the Guifford Wood, No. 477. This chisel has a narrow plate made of rather soft steel, so that it

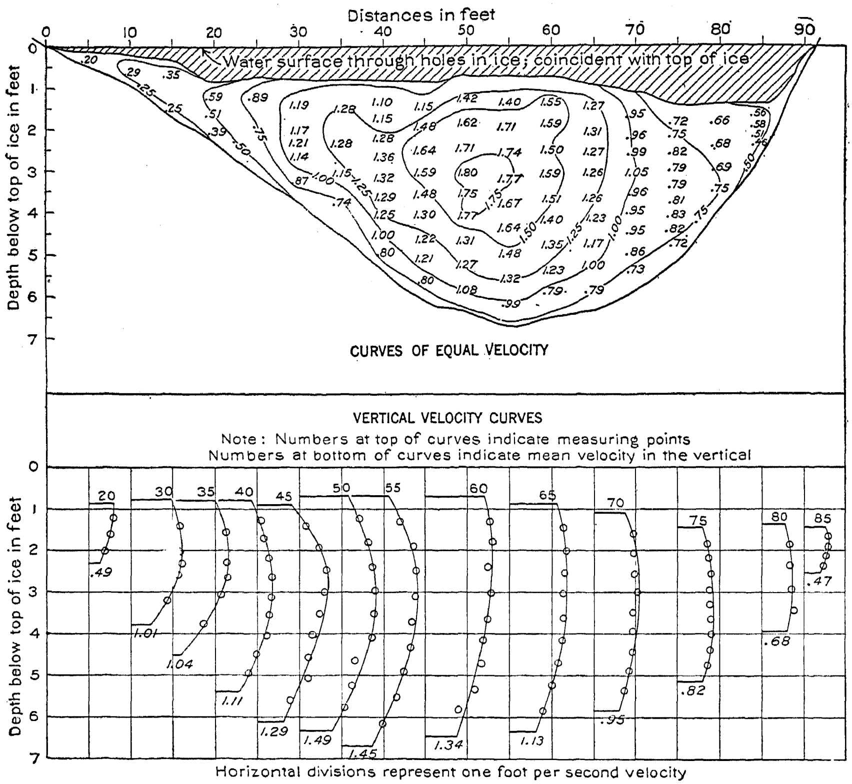


FIG. 18.—Distribution of Velocity under Ice Cover, Cannon River, Welch, Minn.

can be easily sharpened with a hand file. The handle is solid steel, has a ring in the top, and may be joined, if desired, for convenience in carrying. This chisel weighs 14 pounds and is as light as can be satisfactorily used.

The measuring stick should be made of 1 by 1-inch material, graduated to tenths of a foot, and with a 3-inch angle fastened to the zero end. In use this angle is brought against the under surface of the ice and the thickness read at the top surface on the graduated stick.

The accuracy of discharge measurements depends largely on the comfort of the engineer while making the measurement, therefore he should give careful attention to his clothing.

Winter-measurement observations are taken by one of the following methods or by a combination of them:

1. From a bridge or cable, when the stream is clear or not frozen hard enough to support the observer.
2. Through holes cut in the ice.
3. By wading in a section from which the ice has been cleared.

Measurements from a cable or bridge in a section without ice cover are made in the same manner as open-water measurements, except that the meter should not be taken from the water unless absolutely necessary until the measurement is completed. The soundings can be taken by lowering the weight until it touches the bottom and then raising it until the head of the meter is just under the surface of the water, adding to this depth the distance from the top of the meter to the bottom of the weight. If it is necessary to remove the meter temporarily from the water, on its return it should be permitted to run for a period before the revolutions are recorded, as the warmer river water will tend to thaw out any slight congealing in the meter. If the meter is o until it becomes practically rigid it should be thawed and thoro dried near a fire, care being taken that it does not become so hot a melt the rubber connections or the solder on the cups.

The first operation in measurements under ice cover is the cutting of the holes through the ice. The holes should be spaced 5 to 10 feet apart, the interval depending on the width of the stream. On very small streams, where measurements of velocities are desired at many points close together, an entire section may be cut out, thus preventing the greater part of the vertical pulsations of the water. The position of the holes need not be accurately determined before cutting; they should, however, be placed in a straight line and as near as possible at right angles with the current. Round or oblong holes are easier to cut than square holes, and the larger diameter should be parallel to the current. They should be large enough to permit the meter to be easily raised and lowered. A shovel will be found almost essential in clearing away snow from the ice in the vicinity of the holes and removing the chopped ice from the holes. The first 3 or 4 inches of the ice can be cut more quickly with a sharp ax than with the ice chisel, but with the chisel an ax is not necessary. The ice should be cut only at the circumference of the circle, as large cakes can be taken out with less shoveling than the small ones that are made

when the entire cross-section of the hole is chopped. Care should also be taken not to cut through the ice into the water until a few rapid blows of the chisel will clear the entire section. By working carefully holes can be chopped through ice 1 to 3 feet thick with very little splash; if, however, water is standing or flowing on the ice to depths of 1 or 2 inches it is almost impossible to chop holes without getting wet.

As a rule it is advisable to chop one hole through the ice in the center of the section in order to detect the presence of frazil or floating anchor ice. If the hole so cut can not be kept clear from the finer particles of frazil, the measurement will probably give inaccurate results, and the engineer should endeavor to find a better section.

When the holes are cut and cleared of the chopped ice the distances between them should be measured with a steel tape and recorded in the notebook, leaving sufficient space between the recorded measurements for the record of velocities in the vertical. As the winter flow is likely to be fairly uniform, the soundings may be taken independently from the measurements of velocity. The gage height to the water surface should then be read and the soundings taken either with the rod or with the weight and cable. At each hole should be recorded (a) the thickness of the ice, (b) the distance from under surface of ice to water surface, and (c) the total depth of the water. From these data can be computed the depth at which the meter must be placed in each hole in order that it may be at the 0.2, 0.8, or 0.5 position beneath the ice. Thus—

$$0.2 \text{ depth} = (c - b) \times 0.2 + b$$
$$0.5 \text{ depth} = (c - b) \times 0.5 + b$$
$$0.8 \text{ depth} = (c - b) \times 0.8 + b$$

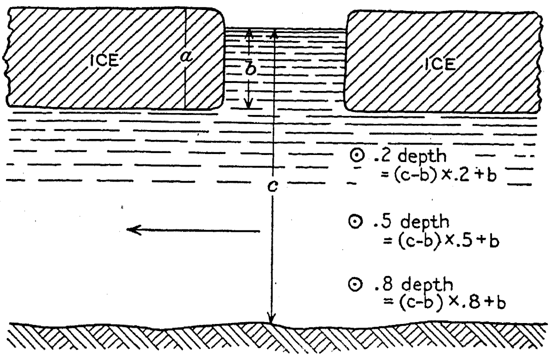
The notation and a form for recording the data are illustrated in figure 19.

The information collected regarding the total thickness of the ice need not enter into the measurement.

When holes are cut through the ice a vertical pulsation, which may amount to nearly half a foot, is often noticed in the water. In measuring the depth of the water, care should be taken to determine the mean. If the depths beneath the ice are greater than 2.5 feet, the measurement should be made by the 0.2 and 0.8 point method; for depths between 1.5 feet and 2.5 feet velocities should be observed at 0.2 and 0.8 and 0.5 depth; for depths less than about 1.5 feet the mid-depth method should be used.

The depths at all measuring points should be measured and the depths at which the meter is to be placed computed before the meter is assembled.

The meter should be so held in the hole that the head is as far upstream as possible to avoid the effect of the vertical pulsations. If a rod is used, the meter can be kept in position by holding the rod against the upstream side of the hole; if a cable, the meter can be held at one position more easily by standing on the cable than by holding it in the hand. The number of revolutions and the time are recorded as in open-water measurements. After the complete data for each observation have been recorded while the meter is still in the water, the meter can be carried quickly to the next hole and the observations continued. In this way, if no frazil is present, the entire measurement may be completed without having the meter in the air long enough for the water on it to congeal.



OBSERVATIONS					
Distance from initial point	Thickness of ice / Water surface to bottom of ice	Total depth of water / Effective water depth	Depth of meter from water surface	Time in seconds	Revolutions
0					
10	a	c	(c-b)×.2+b		
	b	$c-b$	(c-b)×.5+b		
			(c-b)×.8+b		
15					

FIG. 19.—Diagram showing factors used in making discharge measurements and new form proposed.

The section for winter measurements should preferably be chosen during the open season to insure favorable conditions. If the presence of frazil or of floating anchor ice at a section affects more than 10 per cent of the total cross-section, the measurements should, if possible, be made at another section where such conditions do not exist. Sections

at rapids, where the velocities are high enough to carry the ice away quickly, may be most satisfactory, even if the other conditions are not so desirable, and the measurements made at them will be more accurate than those made at a section which is partly clogged with frazil.

MEASUREMENTS IN ARTIFICIAL CHANNELS.

Flow in artificial channels should be studied with especial care on account of the disturbances resulting from the operation of gates and water wheels in power canals and of checks, intakes, and outlets in irrigation canals.

In an irrigation canal the effect of such disturbance usually appears in the control section as a change which destroys the relation of gage height to discharge. Though the conditions may remain satisfactory for making current-meter measurements, estimates of discharge can not usually be made from records of stage and a rating curve.

Changes similar to those in irrigation canals occur also in headraces of power canals. In tailraces the relation of stage to discharge will probably be reasonably permanent, but conditions for measurements of discharge are unsatisfactory, as the distribution of the velocity in the cross-section of the channel is usually not normal and is likely to change with considerable rapidity. Furthermore, the agitation of the water may interfere with the proper action of the meter or other measuring device. Special studies should therefore be made of the conditions existing in each artificial channel to be gaged, and special arrangements of instruments or of channels may be necessary in order to get satisfactory measurements of discharge, particularly measurements in connection with tests of water wheels in place.

FLOAT MEASUREMENTS.

The determination of the discharge of a stream by the float method is comparatively simple and easy, as no delicate instrument is necessary for measuring velocity. The results, however, will not generally be so accurate as those obtained by the current meter.

The first step in making a float measurement is to select and measure the "run" over which the floats are to pass. This run should be 50 to 500 feet long, should be located in a stretch of the stream having a straight and uniform channel, and its ends should be definitely marked on one or both banks by range poles or signals or by tagged lines across the stream.

Floats are placed in the stream above the upper end of the run and allowed to pass over the run, the time in seconds required for their

passage being noted. The quotient obtained by dividing the length of the run in feet by the time of passage in seconds is the velocity in feet per second of that part of the stream traversed by the float. A stop-watch is necessary for the satisfactory determination of the time required by the floats to pass over the course.

The type of float used will depend upon local conditions of channel and current. Tube floats are generally limited in use to artificial channels. Subsurface floats are but little used and only in deep streams. Surface floats are adapted for general use under all conditions.

In an ordinary discharge measurement by this method a number of velocity determinations are made at varying distances from the shore, and the mean of the velocities thus obtained, reduced by a coefficient, is taken as the mean velocity for the cross-section. The magnitude of the coefficient will vary between .85 and .95, the variation depending on the stage and character of the stream.

In more accurate work the distance of the float from the bank will be noted and the mean velocity of the whole section can be determined by plotting the mean position of each float, as indicated by its average distance from the bank, as an ordinate, and the corresponding time of the run as an abscissa. A curve through the points so located shows the mean time of run at any point across the stream. Velocities for each partial area of cross-section are scaled from this curve, reduced to feet per second, and multiplied each by its area to determine partial discharge. The sum of the partial discharges is the total discharge. The coefficient for reducing surface velocity to mean velocity may be applied either to each determination of velocity or to the computed discharge.

The area used in float measurements is the effective area of the section of the river over which the runs are made and is determined by averaging the areas of cross-section of the stream measured at the ends and at intermediate points.

SLOPE MEASUREMENTS.

The mean velocity of a stream has been expressed in the Chezy formula as $V = c\sqrt{Rs}$, in which c is the coefficient combining the total effects of roughness of the bed and all other conditions which may affect the velocity, except the slope and hydraulic radius. This formula has long served as a nucleus about which slope data have been collected, and has been used as a basis for work by Kutter,[a] who developed the following expression for the value of the coefficient c in terms of s, R, and n.

[a] See "The flow of water in rivers and other channels" by Ganguillet and Kutter.

$$c = \frac{41.6 + \frac{.00281}{s} + \frac{1.811}{n}}{1 + \left\{41.6 + \frac{.00281}{s}\right\} \frac{n}{\sqrt{R}}}$$

This, when introduced into the original formula, gives

$$V = \left\{ \frac{\frac{1.811}{n} + 41.6 + \frac{.00281}{s}}{1 + \left\{41.6 + \frac{.00281}{s}\right\} \frac{n}{\sqrt{R}}} \right\} \sqrt{Rs}.$$

In measurements of discharge by the slope method it is necessary to determine (1) the mean area of cross-section, (2) the slope of the surface of the stream, and (3) data in regard to the roughness of the bed, from which to estimate the proper value of n.

In making a measurement by this method a straight channel, 200 to 1000 feet long, must first be selected and measured for the course or "run." In this course the slope and cross-section should be reasonably uniform and the conditions of bed and banks should also preferably be permanent. The slope must be sufficiently large to be measured without a large percentage of error. The effective area of cross-section throughout the run is obtained as the mean of the cross-sections of its ends and intermediate points. The determinations of these cross-sections may be made once for all at a low stage of the stream, if the bed and banks are permanent, or as often as may be necessary for good work if the conditions are changing.

The slope of the surface of the stream is obtained by simultaneous readings of gages placed at the ends of the run, and as it is the important factor in this method, the position of the gages by which it is to be measured should receive careful consideration. Theoretically the gages should be set in the current of the stream, which may, at high stages, be several tenths of a foot higher than water in the same cross-section near the banks. Such location of gages in the current is impossible unless bridges are available to which gages may be attached, and in this case the effect of bridge piers may be sufficient to vitiate the observations. Gages attached directly to bridge piers are not suitable for measurements of slope because of the disturbance of the water around the pier. In locating gages on the banks the exposure should be the same for all gages to be utilized in conjunction for the determination of slope;

otherwise the piling of water against one bank or its recession from another may incorrectly indicate the slopes.

At least two and preferably three gages should be placed in position at the ends (and in the center if three gages are used) of the course in which the slope is to be measured. The datum of each gage must then be accurately referenced by means of a permanent and easily accessible bench mark, and all gages must be connected by levels. As the gages should be read to hundredths, the effect of wave action should be eliminated by the use of some form of stilling box.

The accuracy of the estimates will depend largely on proper placing of the gages, precision in the gage readings, and care in setting the gages to read from the same datum. If the course is not designed for continuous use for slope measurements, reference points from which the elevation of the water may be determined by a single vertical measurement may be used instead of gages.

In collecting data for determining the value of n it should be borne in mind that this factor includes not only the effect of roughness of bed but also that of all obstructions that may retard the water. In general n is larger for the overflow part or parts of the stream than for the channel proper; hence these parts should be treated separately in computing the discharge. For the higher stages of the stream n for the channel proper generally decreases as the stage increases. The engineer must rely largely on judgment and experience in this matter.

The following table gives the ranges of value of n for various types of channels, both natural and artificial. The values for artificial channels are taken from the results of investigations by the Office of Experiment Stations, Department of Agriculture, and by the United States Reclamation Service, and are based on actual conditions of canals in operation. In general, much lower values are found for artificial channels when first constructed than for the same channels after they have been in operation for some time. The values apply for sections on tangents and should be increased if used for curves.

Values of n *in Kutter's formula.*

Character of channel.	*Value of "n."*
Artificial channels.	
Cement, surfaced	.012 to .015
Cement, rough	.015 to .018
Wood, surfaced	.011 to .015
Wood, rough	.015 to .020

Earth, smooth	.017 to .025
Earth, rough	.025 to .030
Cobble	.030 to .035
Vegetation	.035 to .050
Natural channels	
Smooth, sandy, and fine gravel beds	.020 to .025
Rough beds	.030 to .035
Overflow banks with vegetation	.040 to .055

For simplicity in computation, tables giving the values of V and c for various conditions have been published. Among these is Table XV, page 203. Diagrams (Pl. VII) are also used to advantage in this connection.

The slope method is commonly used for estimating flood discharge, often after the crest of the flood wave has passed and when the only data available are the slope and the area of cross-section, as determined from marks along the banks, and a knowledge of the general conditions. Another important present use of Kutter's formula is in the design of canals, for which the slope must be determined in order that the channel may carry a certain quantity of water at a given velocity.

The results obtained by this method are in general only approximate, owing to the difficulty in obtaining accurate measurements of slope and the other necessary data and the uncertainty of the value of n to be used in Kutter's formula.

OBSERVATIONS OF STAGE.

In the collection of records of daily stage of a stream for use with discharge measurements to obtain daily flow, care must be taken to eliminate errors due to the following causes:

1. Change in gage datum.
2. Lack of refinement in gage readings.
3. Inaccuracies of observation by the gage observer.
4. Insufficient readings to give the true daily mean.

The permanence of the original installation of the gage will largely determine the errors due to change in datum. If the gage is properly installed and the gage datum is frequently checked by a level errors from this cause should seldom occur. They are, however, cumulative and therefore specially serious. The importance of the maintenance of gage datum and the precautions in connection therewith have been fully discussed on pages 23 to 36.

Examples for use of diagram.

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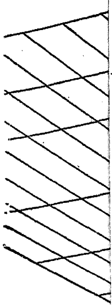

Find.	Procedure.
c	Locate point where radial line $n = .026$ cuts slope curve $s = .00015$; join this point with point on hydraulic radius scale $R = 4.5$. This line cuts the vertical scale of coefficients in $c = 75$.
R	Locate point $n = .026$, $s = .00015$; join this point with $c = 75$. The prolongation of this line cuts hydraulic radius scale in $R = 4.5$.
s	Join $R = 4.5$ with $c = 75$; prolong this line to cut radial line $n = .026$. This intersection falls on slope curve $s = .00015$.
n	Join $R = 4.5$ with $c = 75$; prolong this line to cut slope curve $s = .00015$. This intersection falls on radial line $n = .026$.
V	Find, as in Ex. 1, $c = 75$; draw line joining $R = 4.5$ with $s = .00015$ on vertical slope scale. A line through $c = 75$ parallel to this one cuts velocity scale in $V = 1.95$.
n	Join $R = 4.5$ with $s = .00015$ on vertical slope scale. A line through $V = 1.95$ parallel to this one cuts scale of coefficients in $c = 75$. As in Ex. 4, find $n = .026$.
R	Assume $R =$ say, 5; with $R = 5$, find, as in Ex. 5, $V = 2.10$—showing that assumed R is too large; try $R = 4$, and interpolate. That value of R which makes $V = 1.95$ is the true value.
s	Assume $s =$ say, .001; with $s = .001$, find, as in Ex. 5, $V = 5.0$—showing that assumed s is too large. Try $s = .0001$, and interpolate. That value of s which makes $V = 1.95$ is the true value.

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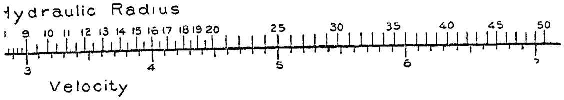


The refinement to which the gage must be read will depend on the sensitiveness of the station. For ordinary stations, equipped with staff gages, readings to the nearest quarter tenth are usually sufficiently accurate and can be readily obtained by the average observer; for canals and small streams it may be necessary to read to hundredths; for large streams, readings to half tenths or tenths may be sufficiently close. Errors due to refinement of readings are generally compensating, but they may be cumulative for considerable periods when the stage is constant. The refinement to which gage heights must be read and used for determining daily discharge depends on the station rating curve and is discussed on pages 104 to 107.

Errors due to inaccuracies of observation by the gage reader may be either compensating or cumulative. They will depend on the honesty and intelligence of the available gage reader and can be guarded against only by careful instructions and close inspection. In many localities it is necessary to install automatic gages on account of the incompetence of available observers. When the automatic gage is used special care must be taken in placing the record sheet, as any errors in setting will introduce cumulative errors in the discharge.

The number of observations to be taken daily in order to obtain true mean daily gage height will depend on the characteristics of the stream and should be determined for each gage station by a series of hourly gage readings, taken at various times throughout the year, as the errors due to this cause may enter only at certain seasons. Errors of this kind will usually be cumulative although under some conditions they may be compensating. For streams whose flow is regulated either naturally or artificially the mean daily stage can be obtained only by an automatic gage; for unregulated streams two gage readings a day will, as a rule, give the mean stage with sufficient accuracy except during flood periods, when additional readings should be taken.

The accuracy of the gage readings will depend largely on the type and character of the gage. All direct-reading gages should be plainly graduated, either to tenths, half tenths, or hundredths, the graduation depending on the refinement to which the gage is to be read, and they should be so placed that they are easily accessible to the observer.

CHAPTER IV.

WEIR STATIONS.

In a broad sense a weir is any artificial structure placed in a stream for the purpose of raising the surface of the water. A weir for measuring discharge must have a well-defined form and a reasonably level crest of permanent shape and elevation, and must not allow a large percentage of the water to pass through, beneath, or around it.

Weirs may be used for measuring the quantity of water in streams because water flows over them in accordance with known, definite laws. They become available for such measurement by the use of formulas in which the quantity of discharge is expressed in terms of the dimensions of the weir and the head of water on its crest, and by coefficients which have been determined by experiments.

Weirs may be divided into two classes—(1) sharp-crested, or standard weirs, and (2) broad-crested weirs or dams, the distinction depending on whether the water in passing over them comes in contact with the crest on a line or a surface. Either of these two classes may be submerged or may have a free overfall—that is, the elevation of the water on the downstream side of the weir may be above or below its crest. Weirs of either of these classes may be vertical or inclined. Usually measurements of flow are made only at vertical weirs having a free overfall, and the following discussion is limited to that class.

In considering the establishment of a weir station, choice must generally be made between a velocity-area station, the use of an existing dam, or the construction of a sharp-crested weir to be used exclusively for obtaining a record of flow. The choice between these types of stations will be governed largely by conditions relating to the cost and accuracy of the records.

SHARP-CRESTED WEIRS.

Sharp-crested weirs used under heads of not to exceed 5 feet offer the best facilities known for determining the flow of streams whose discharges are too great to be weighed or measured in a calibrated tank.

The coefficients for use in formulas for such weirs have been carefully determined for heads under 5 feet, and have a small range in value.

The use of sharp-crested weirs is generally limited by their cost to comparatively small streams or to streams of which very accurate records of flow are desired. They are most commonly employed to divide water among several users, especially for irrigation, and the principal requisite for their location is a site at which the weir can be economically constructed so that there will be no percolation or leakage.

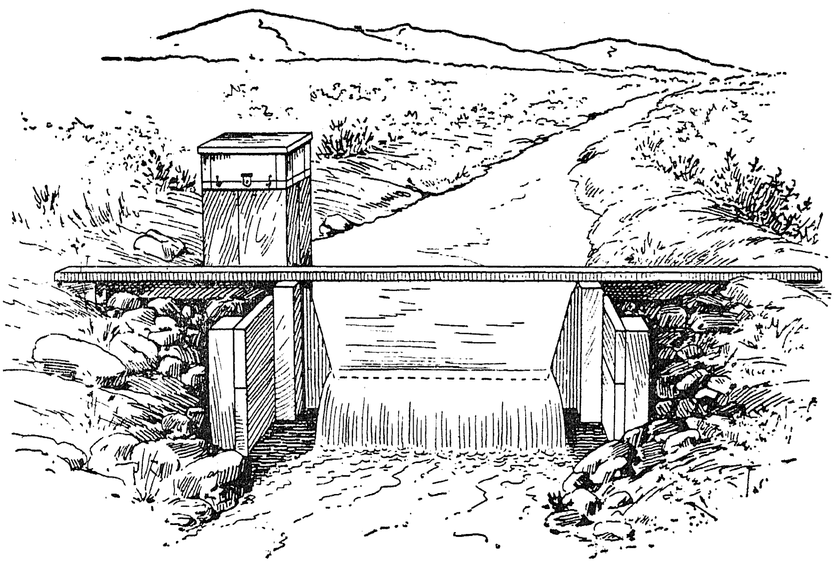


FIG. 20.—Cippoletti Weir, with Water Register in Place.

Sharp-crested weirs may be either rectangular or trapezoidal in form and must have a crest of such dimensions and height that the water will have free fall over it with provision for the admission of air under the overfalling water.

As commonly arranged, the weir projects sharply from both sides and the bottom into the channel conducting the water, thus making the dimensions of the cross-section over the weir less than those of the

channel of approach. This reduction in the cross-section of the channel causes a contraction of the water at the bottom and the ends as it passes over the weir. If both end and bottom contractions exist the weir is called a *contracted* weir. This contraction may be prevented by arranging the channel of approach so that the water is guided both on the bottom and ends directly to the crest of the weir, making what is called a *suppressed* weir. In many weirs the end contractions only are suppressed, when the weirs are said to be partially contracted.

End contractions cause a virtual decrease in the length of crest of the weir. For rectangular weirs this effect is provided for in the formulas. The Cippoletti weir (fig. 20), which is the most common form of trapezoidal weir, is constructed with the outward slope of each end 1 horizontal to 4 vertical. This causes an increase in effective length as the head increases, thus very nearly compensating for end contraction.

In sharp-crested weirs the channel of approach, fore bay, hydrant, or stilling box from which the water flows over the weir must either be sufficiently large to eliminate velocity of approach to the weir, or a correction must be made for such effect in the computations. The structure will therefore vary in size and arrangement for the accommodation of different quantities of water.

BROAD-CRESTED WEIRS.[a]

Weir stations on large streams will usually be located at existing dams which are constructed for purposes of power or navigation, and selection must be made between several available dams or between a dam and a velocity-area station. In either case the advantages and disadvantages of each locality must be carefully considered, as the value of the resulting record of discharge will depend largely upon the possibilities of the station. As compared with velocity-area stations dams may have the advantage of continuity of record through the period of ice but the disadvantage of uncertainty of coefficients to be used in the weir formulas and complications due to diversion and use of water.

In investigating the availability of a dam for gaging purposes, observations must be made concerning certain conditions which are necessary to insure good records. These conditions may be divided into two classes—(1) those relating to the physical characteristics of the dam itself, and (2) those relating to the diversion and use of water around and through the dam.

[a]Stations at broad-crested weirs are fully discussed in U. S. Geol. Survey Water-Supply Papers Nos. 200 and 180.

The physical requirements are as follows: (*a*) Height of dam such that backwater will not interfere with free fall over it; (*b*) absence of leaks of appreciable magnitude; (*c*) topography or abutments which confine the flow over the dam at high stages; (*d*) level crests which are kept free from obstructions caused by floating logs or ice; (*e*) crests of a type for which the coefficients to be used in $Q = clH^{\frac{3}{2}}$ or some similar standard weir formula are known; (*f*) either no flash boards or exceptional care in reducing leakage through them and in recording their condition.

Preferably there should be no diversion of water through or around the dam. Generally, however, part or all of the water is diverted for uses of power or navigation. Such water must be measured and added to that passing over the dam. To insure accuracy in estimates the water diverted must be reasonably constant in quantity, and so utilized that it can be measured either by a weir, a current meter, or through a simple system of water wheels which are of standard make or have been rated as water meters under working conditions and so installed that the gate openings, heads under which they work, and their angular velocities may be accurately observed.

The combination of physical conditions and uses of the water should be such that the estimates of flow will not involve, for a critical stage of considerable duration, the use of a head on a broad-crested dam of less than 6 inches. Moreover, when all other conditions are good, a careful observer is still essential in order to obtain reliable results.

The field work for the establishment of a station at a dam must be sufficient to provide for obtaining the records of gage height, and must also include the surveys and the collection of information which will make possible the correct interpretation and application of these records in the computation of discharge. It must consist, therefore, of the establishment of a gage for determining the head on the dam, and, if water is diverted through a head race and used through wheels or wasted through gates or over weirs, the establishment of sufficient other gages for determining the effective head on such turbines, gates, or weirs. Provision must be made for the systematic reading of these gages as well as for recording the conditions of wheel-gate openings, speed of wheels, elevations of crests of adjustable waste weirs, openings of waste gates, and elevation and conditions of flash boards. The gages must each be referenced by a convenient bench mark and all connected by a line of levels. An instrumental survey of the dam must be made to determine the length, profile, and cross-section of the crest. Cross-

sections of the channel of approach to the dam should also be measured in order to estimate the velocity of approach. Usually special forms for records and computations must be prepared for each such station.

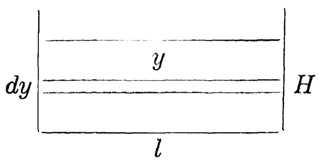
WEIR FORMULAS.[a]

The discharge over a weir is the product of the area of effective cross-section of the vein of water passing over it, the mean velocity in this area, and a coefficient determined experimentally, which varies with the form of the weir. The area of cross-section is determined approximately by the length of the crest and the head or the depth of water on it. The velocity is determined approximately by the head. These two quantities, length of crest and head, together with the coefficient, are therefore factors entering all weir formulas. They must, however, be modified for differences in forms of weirs, conditions of contraction, and velocity of approach.

The effects of end contraction and velocity of approach are allowed for in the formula by modifying the length of the crest and head respectively, or in the coefficient. The coefficient to be used in any instance must have been determined for that particular formula.

FUNDAMENTAL FORMULAS.

The fundamental formula for rectangular weirs may be derived by the calculus as follows:



$$Q = c\int_{0}^{H} \sqrt{2gy}\; l\,dy = \tfrac{2}{3}\, cl \sqrt{2g}\, H^{\frac{3}{2}} \qquad (1)$$

in which l represents the length of the weir; H, the head of water on the weir; y, the head on any horizontal strip of differential width, dy; g, the acceleration of gravity; and c, coefficient that must be determined experimentally and that varies with different conditions of crest, channel of approach, etc. In the integral expression, $\sqrt{2gy}$ is the theoretical velocity of the water in the strip whose area is ldy. The integration between the limits 0 and H of the products of the infinitesimal areas by the velocities through them gives the total discharge, Q.

Modifications are made necessary, as previously explained, by reason

[a] See Hydraulics, by Hamilton Smith, jr.

of velocity of approach, variations in contraction of the water as it passes the weir, or variations in form of weir. If the end contraction is perfect, it causes at each end of the weir a shortening of the effective length by approximately .1 H.

If allowance is made for such end contraction, formula (1) becomes

$$Q = \tfrac{2}{3}\, c\,(l - .2H)\ \sqrt{2g}\, H^{\frac{3}{2}} \tag{2}$$

The same results are also accomplished in a different way by modifying properly the coefficients used in formula (1).

The velocity of approach "V" causes a virtual increase in head. The magnitude of such increase is the head corresponding to that velocity, is represented by h, and equals $\frac{V^2}{2g}$. Such velocity of approach may be obtained approximately as the quotient by dividing the discharge by the area of cross-section of the channel of approach. The result so obtained should, however, be multiplied by a coefficient greater than unity (usually assumed to be between 1 and 1.5). Since the amount of the discharge is the quantity to be determined, the approximate value of V must be found from an approximate determination of Q by an application of the weir formula, neglecting the velocity of approach.

The correction for velocity of approach may be effected by adding h directly to the measured head in formula (1), as follows:

$$Q = \tfrac{2}{3}\, cl\ \sqrt{2g}\ (H + h)^{\frac{3}{2}} \tag{3}$$

or the correction may be applied before integration as follows:

$$Q = c\int_0^H \sqrt{2g\,(y + h)}\ l\,dy = \tfrac{2}{3}\ cl\ \sqrt{2g}\left[(H + h)^{\frac{3}{2}} - h^{\frac{3}{2}}\right] \tag{4}$$

Formulas (3) and (4) are both in common use.

The formulas already explained—(1), (2), (3) and (4)—serve as bases for the formulas of all free overfall rectangular weirs, whether sharp or broad-crested. Values of c have been determined for use in each of these formulas for various types of weirs. Many sets of coefficients are therefore available, but each is applicable only to its formula.

RECTANGULAR WEIRS.

The formulas shown above, or slight modifications of them, are in general use for rectangular weirs. Of the modifications in common use, the *Francis formula* (5) is the simplest in form and application.

Francis determined that for a suppressed weir, without velocity of approach, c had an average value of 0.62. The product of the three constants of formula (1), 0.62, $\frac{2}{3}$, and $\sqrt{2g}$ = 3.33, thus making

$$Q = 3.33\ lH^{\frac{3}{2}} \tag{5}$$

The discharge per foot of length as determined by this formula is given in Table II, pages 190, 191.

When modified to allow for end contractions and velocity of approach, formula (5) becomes

$$Q = 3.33\ (l - .2H)\ \left[(H + h)^{\frac{3}{2}} - h^{\frac{3}{2}}\right] \tag{6}$$

If there is no velocity of approach formula (6) becomes

$$Q = 3.33\ (l - .2H)\ H^{\frac{3}{2}} \tag{7}$$

Table I, pages 188, 189, shows the discharge determined by formula (7).

In the use of formulas (5), (6), and (7), the dimensions must always be expressed in feet, because that unit has been introduced in the value of g, which appears in the coefficient.

Other formulas in common use were devised by *Bazin*, among which is the following, which gives the discharges for a sharp-crested weir without end contractions:

$$Q = \left(0.405 + \frac{.00984}{H}\right)\left(1 + 0.55\frac{H^2}{(p + H)^2}\right) lH\sqrt{2gH} \tag{8}$$

in which H = observed head in feet; p = height of weir in feet; l = length of crest in feet; Q = discharge in second-feet.

Table IV, pages 194–196, shows the discharge as computed from formula (8).

TRAPEZOIDAL WEIR.

The trapezoidal weir is unimportant, except the Cippoletti weir (fig. 20), in which the outward slope of the ends (p. 83) counteracts the decrease in length due to end contractions. Ordinary formulas for suppressed weirs are therefore approximately applicable to it. Table II may be used for the Cippoletti weir with an error of about 1 per cent, giving results too small by that amount. Special determinations of coefficients for this weir have, however, been made and the resulting formula for discharge without velocity of approach is

$$Q = 3.3\tfrac{2}{3}\ lH^{\frac{3}{2}} \tag{9}$$

BROAD-CRESTED WEIRS.

Coefficients for several types of broad-crested weirs have been determined by Bazin, in France, and under the direction of Prof. Gardner S. Williams at the Cornell University Testing Laboratory, for the United States Deep Waterways Commission, by Mr. John R. Freeman and by members of the United States Geological Survey. The results of all of these experiments have been brought together by Mr. R. E. Horton in Water-Supply Paper No. 200, United States Geological Survey.

In ι .s paper it is shown that within certain limits of head the discharge over several types of broad-crested weirs may be found by the use of formula.

$$Q = 2.64\, lH^{\frac{3}{2}}. \tag{10}$$

This formula is applicable to broad-crested weirs of any width of cross-section exceeding 2 feet within such limit of head that the nappe does not adhere to the downstream face of the weir for low heads nor tend to become detached from the crest with greater heads. If the latter condition exists, the coefficient increases to a limit near the value which applies for a thin-edged weir, a point being finally reached where the nappe breaks entirely free from the broad crest and discharges in the same manner as for a thin-edged weir. Formula (10) may be applied safely to any weir having a crest width exceeding 2 feet and with heads from 0.5 foot to 1.5 or 2 times the breadth of weir crest. Table III, pages 192–193, shows the discharge as determined by this formula.

From the experiments mentioned above Mr. E. C. Murphy has developed multipliers to be used in connection with Bazin's formula for discharge over a sharp-crested weir to find the discharge over a broad-crested weir. Tables V, VI, and VII, pages 197–199 and fig. 39 show these multipliers and the forms of weirs to which they pertain. These tables are to be used in connection with Table IV, which has been made the basis in their computation.

COMPUTATIONS.

In the computation of discharge over a weir, whether sharp or broad-crested, a rating table is first prepared which gives the discharge for the various heads occurring during the period of observation. This rating table is computed by substituting values of head, dimensions of weir, and coefficients depending upon the type of weir under consideration in the formula applicable to such weir.

Many dams, unless built of solid masonry, have irregular crests due to unequal settlement. Such a dam must be divided into parts, each of which has a uniform elevation of crest, the formula applied to each part independently, and the results combined to form the rating table.

If any fixed condition of flash boards or other modification of the crest of the dam exists for a considerable period of time, a similar rating table should be made also for such condition. In the same way it will be found to be economical to compute rating tables for any fixed waste weirs in the head-race and for the usual condition of waste gates, wheel gates, etc., which are sufficiently constant to warrant such computations.

With rating tables at hand as above described, the computations of daily discharge are made by entering each rating table for the partial discharge through or over the structure for which that table has been made. The partial discharges so obtained are summed to give the total rate of flow. Discharges at stages and for conditions which are not covered by the rating tables must be computed independently.

These rating tables are as a rule instrumental in saving time in the computations, but their principal value arises from the fact that errors are much less likely to appear in the results than if each discharge is computed independently from a formula. For the same reason tables are more satisfactory than diagrams.

CHAPTER V.

DISCUSSION AND USE OF DATA.

COMPUTATION OF DAILY FLOW.

Certain analyses and computations must precede full use of field data in regard to stream flow. The first step in such analyses is the determination of the daily flow, which is computed in terms of second-feet and is the basis for all future deductions and discussions.

The daily discharge of a stream may be computed in various ways, the method to be chosen depending on the method used in making the discharge measurements — whether by weir or by the velocity-area method. Computations for weir stations are described in Chapter IV.

The base data necessary for the computation of daily flow for velocity-area stations are:

(1) The results of occasional discharge measurements,

(2) Records of gage height, and

(3) Descriptions of conditions at the gaging station.

The methods adopted depend on the control section (see pages 45 and 46)—whether permanent, shifting, or affected by ice as described on the following pages.

GAGING STATIONS WITH PERMANENT CONTROL.

For stations on streams with permanent beds it is possible to prepare, from the data collected, station rating tables, each of which gives for its station the discharge which corresponds to any stage of the stream, and which, when applied to the daily gage heights, gives the daily discharge. The basis for a station rating table is a rating curve which shows graphically the discharge corresponding to any stage of the stream and is usually constructed by plotting the results of the various discharge measurements with gage heights as ordinates and discharges as abscissas. These points define the curve, which is then drawn by use of French curves. The rating table is then determined from the rating curve by tabulating the discharges for each tenth change in stage, adjusted by taking first and second differences.

If accurate and well-distributed discharge measurements covering the range of stage are available, the station rating curve can be readily constructed. Frequently, however, the measurements are more or less discordant and do not cover all stages. As a result special studies are necessary to determine the relative accuracy of the measurements and the position of the curve.

Since the discharge is the product of two factors—the area and mean velocity—any change in either factor will produce a corresponding change in the discharge. The curves of area and mean velocity furnish, therefore, valuable assistance in studying the accuracy of the measurements and in determining the true position of the rating curve. These curves are defined by plotting gage heights as ordinates, and area and mean velocity, respectively, as abscissas.

Curves of area and mean velocity can be constructed only when the channel, both at the control and the measuring section, is permanent. If the control remains permanent, the rating curve for the station will be well defined, even though shifts at the measuring section may make the construction of curves of mean velocity and area impossible. If the control is permanent the effect of changes in area will be counteracted by changes in velocity, thus making no change in the rating curve.

AREA CURVE.

The curve of area shows the relation between the gage height and the area of the cross-section of the stream. This area must include both moving and still water in order to be useful for comparison. Two factors, the width and depth, or gage height, govern the form and position of the curve, which is normally concave to the X axis but may, under special conditions, be straight. For ordinary conditions, where the width increases with the stage, the curve may be assumed to be a series of parabolic arcs whose parameters vary with the slope of the banks. If the banks are vertical the increment is constant and the curve becomes a straight line. It is never concave to the Y axis unless the unusual condition of overhanging banks exists.

The area curve can always be definitely drawn from a careful series of soundings, which should be taken at low water, during the period over which the discharge curve is to apply, and be developed to high water by use of a level. The curve can be constructed easily, and generally with sufficient accuracy, by determining the area only at those gage heights at which the slopes of the banks change. If extreme accuracy is desired the area should be computed for each half-foot of

gage height. High-water soundings and those made in deep streams in which the velocity is great are liable to large errors, and areas computed from them should be carefully scrutinized. Such soundings have been more prolific of sources of error in discharge measurements than all other factors combined.

Since for an infinitesimal change in stage the increase in area equals the product of the width at that stage by the difference in gage height, it follows that the width equals the quotient of the increase in area divided by the difference in gage heights, which ratio is the tangent of the angle that the area curve makes at that stage with the vertical; therefore the direction of the area curve for any stage is determined by

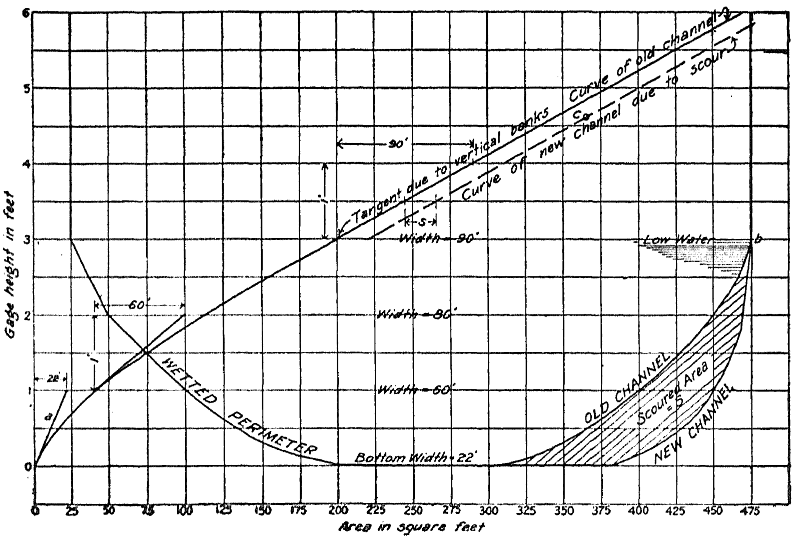


FIG. 21.—Typical Area Curves, Illustrating Their Form.

plotting from the vertical the angle whose tangent is the width at that stage. As most area curves are distorted by magnifying the vertical scale, the principle is utilized by laying off unity on the vertical or gage-height axis to the scale of gage heights, and the width on the horizontal or area axis to the scale of area (fig. 21).

Such curves when referred to origins of coordinates at the elevation of the lowest point in the cross-section exhibit the following useful characteristics: (*a*) For all sections except those with flat bottoms the area curve becomes tangent to the *Y* axis at the origin; (*b*) if the bottom is flat the curve cuts the *Y* axis at the origin at an angle whose tangent is the width of the bottom (*a*, fig. 21); (*c*) if the banks are vertical the

increment is constant and the curve proceeds in a straight line (fig. 21); (*d*) the area curve is permanent in curvature for all gage heights above the plane below which all shifts occur.

The accuracy of the areas as measured at the time of discharge measurement may be quickly tested by plotting them and drawing through each a straight line whose direction is tangent to the curve at that gage height and is determined by the width of the stream, as explained above. The curve should then be drawn from mean low water and kept parallel to the tangents at each point. Errors and discrepancies are at once

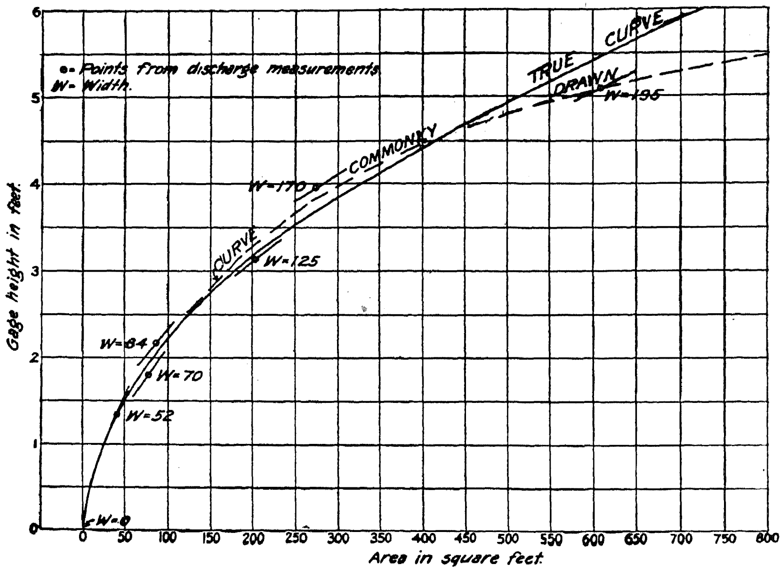


FIG. 22.—Typical Area Curves, Illustrating Their Construction.

apparent (fig. 22). The abscissas between the plotted points and the curve show the error resulting from the combination of errors in computation and soundings, and from changes in channel.

At stations where the banks of the stream are practically permanent, changes in section, if any, take place usually below the low-water line. If the area of such a section changes, the part of the curve above low water, which has been accurately constructed, may be shifted a proper distance horizontally to the right or left and be made to show accurately the areas of the new cross-section (fig. 21). The constant abscissa length between the old and new position of the curve is the algebraic sum of the changes in the area of the section, + for gain in area by

scour and — for loss in area by fill. A single determination of area at any gage height above low water therefore determines the new position of the curve, c (fig. 21).

MEAN VELOCITY CURVE.

As stated in Chapter III, the mean velocity of the stream is the average rate of motion of all the filaments of water of the cross-section and depends principally upon (1) the surface slope of the stream, (2) the roughness of the bed, and (3) the hydraulic radius, and has been expressed in the Chezy formula as $V = c\sqrt{Rs}$, in which the coefficient c has been expressed by Kutter in terms of s, R, and n.

Since slope is the most important factor affecting velocity, when the rate of change in the slope is rapid the velocity tends to follow such change. When the slope becomes constant, changes in the velocity are controlled by the other two factors. the hydraulic radius and the coefficient of roughness.

The curve of mean velocity shows the relation between the gage height and the mean velocity of the current in the measured section. It is located by means of points which are determined by plotting the gage heights and corresponding mean velocities as coordinates. If sufficient measurements have been made to define the curve throughout the range of stage, no further investigation of its properties will be necessary. It frequently happens, however, that the curve must be constructed from limited or discordant values of velocity. To do this intelligently and satisfactorily a knowledge of the properties of the curve under various conditions of flow is essential, and in such cases it is advisable to develop the curves of R and s.

For usual conditions of natural flow in which the stage of no flow is the lowest point in the measured section, the mean velocity curve has approximately the form of a parabola with axis vertical and origin at or below the bed. It approaches a straight line, however, for the higher stages.

When the gaging section is in a stretch of the stream where zero flow occurs with ponded water at the section of the gage, the mean velocity curve will reverse at low stages and approach zero convex to the gage axis. The degree of curvature and the point at which the curve reverses are apparently governed chiefly by the amount of ponded water at the gage, the roughness of the bed, the form of the controlling bar, and other channel conditions. If the stream is small and shallow the change in direction is more abrupt. This peculiar reversal is probably

due to the rapid rate of change of the slope at extreme low flow. At zero flow the slope is of course zero. The least flow causes a slope of the surface and this slope increases with the stage, up to a certain point.

Three methods of extending the mean velocity curve from medium stages to high water have been employed: (1) Extend the curve as a tangent from the last observed value; (2) extend the curve as a hyperbola, i. e., approaching the straight line as its asymptote; (3) assume the slope constant or increasing slightly over the intermediate stages and compute the value of the velocity from the formula $V = c\sqrt{Rs}$, using the most probable value of the coefficient of roughness.

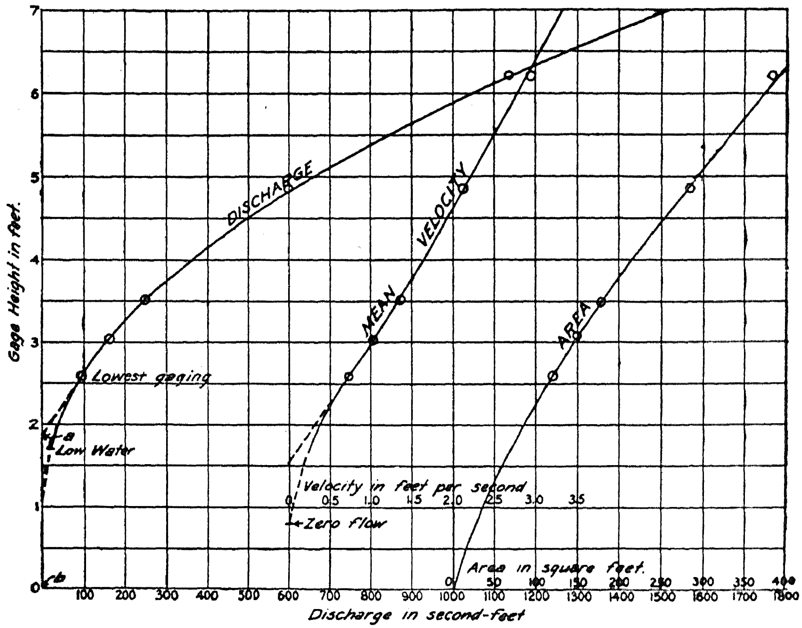


FIG. 23.—Typical Rating Curve, Showing Low-Water Extension.

The curve should be extended into low water with the greatest care. A slight variation from the true direction of the curve means a large percentage of error in the resulting estimate of minimum discharge. All conditions at the station should be studied. The curve must always be drawn to intersect the Y axis at the gage height of zero flow. If the point of zero flow is not known its true position will lie between the gage height of the bottom of the channel and the point where the tangent to the discharge curve at its lowest known value cuts the Y axis, as between a and b, fig. 23). If the mean velocity curve intersects the axis above the gage height of the bed of the stream—that is to say, if

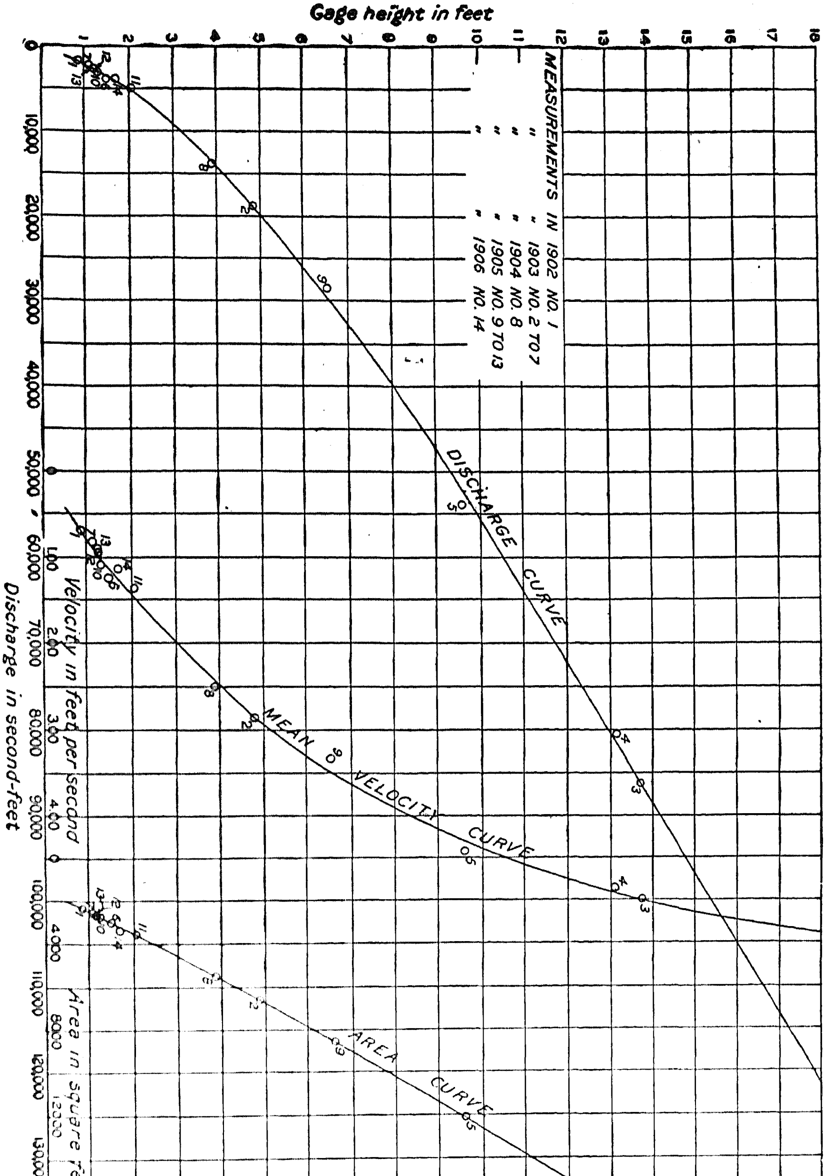


Fig. 24.—Discharge, Mean Velocity, and Area Curves, Potomac River at Point of Rocks, Md.

there is ponded water—the curve will be convex to the Y axis; if it cuts the axis at the gage height of the bed of the stream the curve will be concave to the Y axis (fig. 23).

When measurements are not made at the gage—for example, when low-water measurements are made by wading—the discharge should be divided by the area of the section at the gage and the resulting velocity plotted on the velocity-curve. Points so found are useful in extending the velocity-curve into low water.

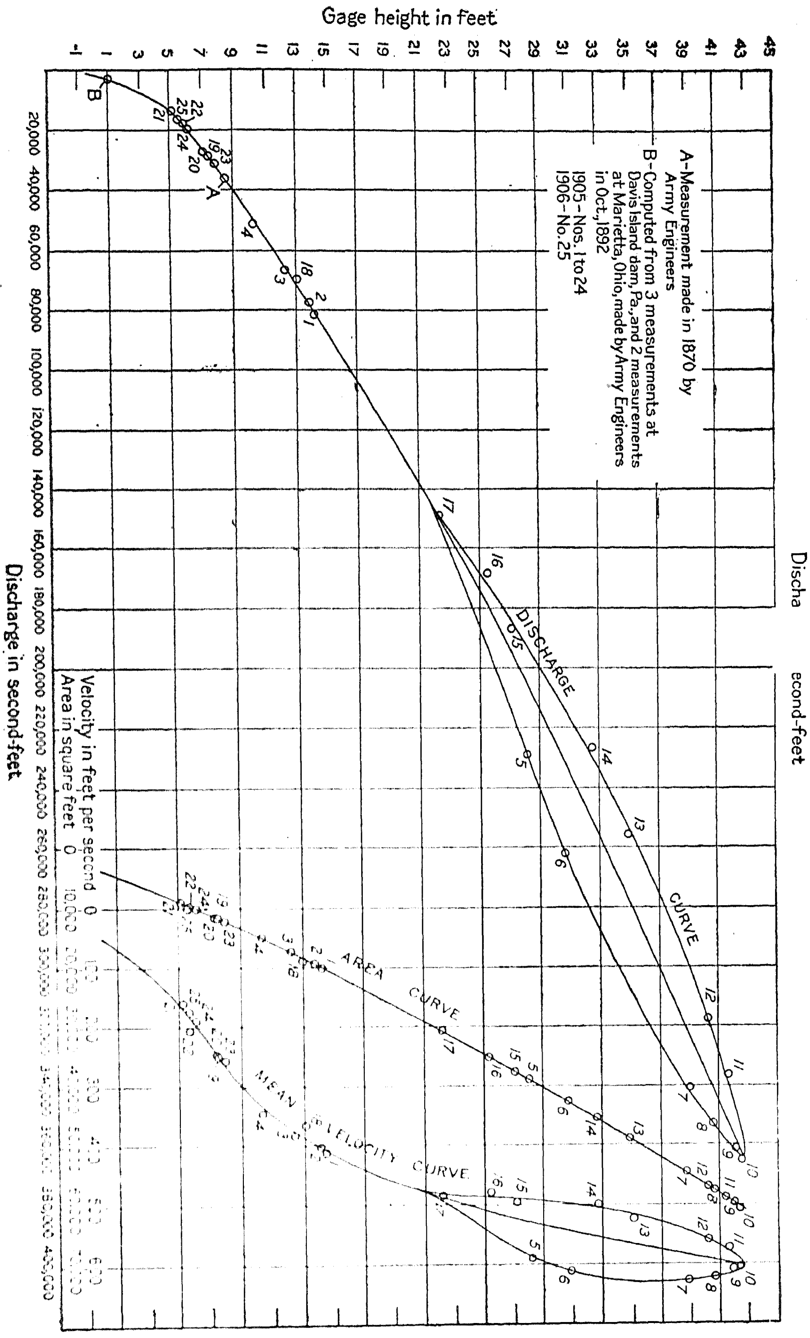
When the current is diagonal to the measured section the observed velocities should be reduced to velocities at right angles to the measured section, but the area should not be reduced. The area is a measured quantity, while the angle of the current is usually estimated and often varies with the stage.

STATION RATING CURVE.

Station rating curves which show graphically the discharge corresponding to any stage of the stream may be plotted either on ordinary or logarithmic cross-section paper. When ordinary cross-section paper is used the measurements of discharge are plotted either with discharge and gage heights as coordinates or with discharge and $A \sqrt{d}$ as coordinates, in which A is the area and d is the mean depth of the cross-section. When logarithmic cross-section paper is used, discharges and gage heights are the coordinates.

Ordinary cross-section paper with discharge and gage height as coordinates.—The usual method of constructing a rating curve for a gaging station is to plot the results of the discharge measurements on ordinary cross-section paper with gage heights and corresponding discharges as coordinates (fig. 24). The points so located define the position of a curve which is drawn among them. The horizontal and vertical scales will be so chosen that the curve may be used within the limits of accuracy for the work, and in its critical portions will make, as nearly as may be, angles of 45° with each axis. The discharge curve under natural special conditions, due to change in control, it may reverse at high stages and become concave to the gage axis.

If a sufficient number of accurate discharge measurements are available and are distributed throughout the range of stage, the discharge curve may be readily and accurately constructed. When such measurements are not available curves of reasonable accuracy may frequently be constructed by use of area and mean velocity curves or by one of the other methods of plotting.


Gage height in feet
A-Measurement made in 1870 by Army Engineers
B-Computed from 3 measurements at Davis Island dam, Pa., and 2 measurements at Marietta, Ohio, made by Army Engineers in Oct., 1892
1905-Nos. 1 to 24
1906-No. 25
DISCHARGE CURVE
AREA CURVE
MEAN VELOCITY CURVE
Velocity in feet per second
Area in square feet
Discharge in second-feet
Discha
econd-feet

In order to determine the accuracy of the individual measurements used in locating the station rating curve it is necessary to plot, as a function of the gage height, not only the discharge but also the mean velocity and area for each measurement. In this plotting the same gage-height scale should be used. The true area curve and approximate curves of discharge and mean velocity are then drawn through the points. The relation of the plotted points of discharge to the rating curve will show any discordant measurements. Whether the discord is due to erroneous area or velocity determinations will be shown by the relation of these respective points to the area and velocity curves, and the cause of any discrepancies in either factor can then be investigated. Such discrepancies may arise from error of observation or of computation.

The relative accuracy of the various plotted discharges having been determined, the rating curve can then be drawn, due weight being given to the various measurements. Points for portions of the curve not defined by actual discharge measurements can be determined by multiplying the area by the mean velocity as measured from the curves of area and velocity. For extending the rating curve either above or below the limits of the measurements the mean velocity and area curves may be constructed, as previously described, to the stages for which the discharge curve is desired, and the discharge curve found by taking the product of the two curves.

Whatever the method adopted in drawing the rating curve it should always be checked by computing the curve of mean velocity from the curves of area and discharge. If the curve of mean velocity so determined is not consistent with conditions at and near the station the discharge curve should be revised.

The discharge at a given stage of a rapidly rising stream is larger than for a falling or stationary stream at the same stage, as the surface slope, and hence the velocity, is greater for the first condition. This effect is but little noticed except during periods of extreme high water. At such times the water passes down the stream in a flood wave, and after the crest is passed a retarding effect may be caused which may reduce the slope practically to zero.

The curves shown in fig. 25 illustrate this. They are based upon the table of measurements on page 101. Therefore, in studying the plotted measurements, the fact whether the stream is rising, falling, or stationary should be considered. Inasmuch as rising stages are of much shorter duration than falling or stationary stages, more weight should

be given to measurements made on falling or stationary than on rising stages.

Discharge measurements of Ohio River at Wheeling, W. Va. Made in 1905 by E. C. Murphy.

No.	Date.	Area of section.	Mean velocity.	Gage height.	Change of stage.[a]	Discharge.
		Sq. ft.	*Ft. per sec.*	*Feet.*	*Feet.*	*Sec.-ft.*
5	March 20	38,890	5.89	28.2	+.68	229,200
6	" 20	42,750	6.13	30.8	+.60	261,900
7	" 21	54,780	6.23	38.9	+.37	341,100
8	" 21	57,360	6.18	40.7	+.20	354,400
9	" 22	59,580	6.07	42.05	+.05	361,600
10	" 22	60,510	6.05	42.5	+.05	365,700
11	" 23	58,830	5.73	41.6	−.20	336,900
12	" 23	56,790	5.60	40.3	−.27	318,100
13	" 24	49,250	5.20	35.2	−.35	255,800
14	" 24	45,550	4.99	32.7	−.40	227,300
15	" 25	37,560	4.95	27.2	−.23	186,100
16	" 25	35,050	4.80	25.5	−.14	168,100
17	" 27	30,830	4.83	22.44	−.05	149,100

[a] Rate of rise or fall per hour; rising +; falling —.

As the mean velocity and area curves, which are factorial curves in making the station rating curve, do not under ordinary conditions follow any mathematical law, the discharge curve will not generally be a mathematical curve. For ordinary streams it is made up of a series of parabolas. For many streams it approaches very nearly the form of a single parabola. Some engineers construct the rating curve by mathematical treatment, by use of least squares. In ordinary practice, however, this is not considered practicable, as the graphic method can be used with greater ease and speed and gives results as close as the data will justify.

If the engineer is familiar with the conditions in the channel at and near the station, a few careful measurements, well distributed, may serve to define the curve of mean velocity. If slope observations are taken and the point of zero flow is determined, a very good approximate rating can be made from two or three measurements.

Ordinary cross section paper, with discharge and $A\sqrt{d}$ as coordinates.—In the construction of a rating curve based on a limited number of measurements, it is evident that it is much safer to extend a straight line than a curve. Investigations have consequently been made of the component parts of the discharge curve for a quantity which is readily measurable, and to which the discharge is approximately proportional, for use in conjunction with the discharge as a coordinate for plotting the discharge curve. The area times the square root of the mean depth of the stream, $A\sqrt{d}$, has been found by J. C. Stevens to be such a quantity.

From Kutter's formula $Q = Ac\sqrt{Rs}$ may be written $Q = (A\sqrt{R})(c\sqrt{s})$. If $(c\sqrt{s})$ is constant or approximately so, then Q varies directly as $(A\sqrt{R})$, and consequently when these two quantities are plotted as coordinates the result is a straight line. c is a function of s, R, and n. R increases with the stage. It is also a matter of observation that s in general increases with the stage, the relative change being small for high stages. For comparatively large slopes the effect of s on c is insignificant, or, to quote Trautwine, "for slopes greater than .01 the coefficient c is the same as for that slope." For flat slopes s has an appreciable effect on c. For a value of R greater than 3.28 feet or 1 meter, c

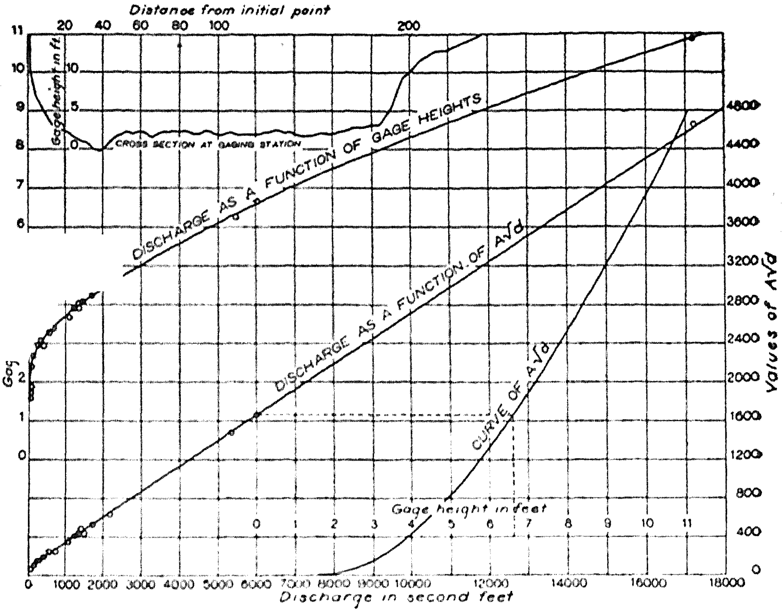


FIG. 26.—Rating Curve showing Discharge as a Function of $A\sqrt{d}$.

and s vary inversely, while c is of itself a decreasing function of s and an increasing function of R. Hence the product of $c\sqrt{s}$ may remain practically constant for a given set of conditions, but for values of R less than 3.28 feet, c is an increasing function of both s and R, and hence the product of $c\sqrt{s}$ is not a constant. The value of this method lies chiefly in making estimates for the higher stages and is not so generally applicable to shallow streams.

Based on the above conditions and assumptions, discharge curves may be plotted with Q and $A\sqrt{R}$ as coordinates. It has been found, however, that d, the mean depth of cross-section, can be substituted

for R and give practically the same results in plotting. It is also easier to determine d than R.

In the application of this method (fig. 26) plot the elevation of the bed of the stream above gage datum and thereby obtain a cross-section. From the cross-section prepare a table giving widths, areas, mean depths and values of $A\sqrt{d}$ for each foot or half-foot of gage heights. Widths may be scaled directly from the cross-section. The table of areas is quickly prepared by first computing the area for one gage height about midway of the range of stage. For increasing gage heights add successively the areas of trapezoids formed by the widths and gage-heights interval. For decreasing gage heights subtract these successive areas.

After the table of areas has been prepared the quantities $A\sqrt{d}$ $\left(\text{or } A\sqrt{\frac{A}{w}}\right)$, where w = width, can be read directly with a slide rule. On cross-section paper draw the curve of $A\sqrt{d}$, using gage heights as abscissas, as shown in the diagram. After this curve is drawn the values of $A\sqrt{d}$ are no longer required. Lay off a scale of discharge as abscissas To plot a discharge measurement project from the horizontal scale of gage heights to the curve of $A\sqrt{d}$, thence horizontally to intersect the given discharge as indicated by dotted line. Points so plotted will generally conform to a straight line.

The illustration (fig. 26) also shows the station rating curve, in which the same scale of discharges is used with gage heights as ordinates, shown on the left.

The straight line marked "discharge as a function of $A\sqrt{d}$" does not pass through the origin for reasons elsewhere stated as to the effect on the coefficient c of the rapidly changing slope at this stage. Therefore, when but a single measurement is at hand the line should be drawn to intersect the scale of $A\sqrt{d}$ at some point above the origin. This point has been found to correspond approximately to the gage height at which the mean depth of flowing water is between 1 and 2 feet.

In the case, frequently encountered, where there is ponded water at the gage height of zero discharge, the corresponding value of $A\sqrt{d}$ should be subtracted from the tabular values of this quantity before plotting. The gage height for which the discharge is zero can be determined by a careful examination, with levels or soundings, of the bed below the gaging section. Even in this case the straight line discharge curve will pass above the origin and should be treated as above outlined for conditions where ponded or dead water does not exist.

Logarithmic cross-section paper.—Cross-section paper graduated logarithmically may also be used in plotting the rating curve. On this

paper discharges and gage heights are plotted as coordinates. The curve resulting from the points so plotted is practically a straight line and has a corresponding advantage for extension. The use of logarithmic cross-section paper is fully discussed on pages 145 to 153.

RATING OR DISCHARGE TABLE.

After the station rating curve has been constructed the next step in the computation of daily discharge is to prepare the station rating table, which gives the discharge of the stream at any stage. This table (see page 108) will be constructed either for tenths, half-tenths, or hundredths of gage height, according to the readings of the gage to which it is to be applied. The table is made by first taking the discharges for various gage heights directly from the station rating curve. These discharges are then so adjusted that the differences for successive stages shall in general be either constant or gradually increasing.

APPLICATION OF RATING TABLE TO GAGE HEIGHTS.

In the application of a discharge rating table to gage heights for obtaining the daily flow of a stream, it is necessary to consider, first, the frequency of gage heights to be used, and second, the refinement with which they should be used.

ency of gage heights.—Theoretically, the mean daily discharge of a stream is the mean of the discharges for every second during the day. In ordinary computation of daily flow, it is assumed that the rate of discharge throughout the day varies so little or with such regularity that the daily discharge may be determined by entering a rating table with a mean daily gage height obtained either from a few observations or from a continuous record made by a water stage register. As the discharge is in general an increasing curvilinear function of the gage height, the use of a mean daily gage height with a rating table gives a result that is always too small. On the magnitude of this error, which will vary with the curvature of the rating curve and with the daily range in stage, will depend whether the daily discharge can be obtained directly by a mean daily gage height or by averaging the discharges corresponding to gage heights for shorter intervals. Hourly discharges are frequently used. As an ultimate limit the absolute mean discharges for the day may be obtained by a discharge integrator which operates on the principle of a planimeter and contains as an essential element the rating curve of the station. Such an integrator has been developed by Mr. E. S. Fuller, Assistant Engineer, U. S. Geological Survey.

It is necessary, therefore, to study each gaging station, in order to choose the frequency with which the gage heights should be applied to

the rating table. In such an investigation a maximum allowable error of 1 per cent is assumed. The amount of daily range in stage allowable at a given mean daily stage, in order not to introduce errors due to curvature of the rating curve in excess of 1 per cent, can be found graphically by constructing a chord to the rating curve such that the horizontal distance, measured by the discharge scale, from its middle point to the curve equals 1 per cent of the discharge at the gage height of the middle point. The difference in gage height at the ends of the chord will be the allowable daily range.

A table of such limits covering the range of stage, used with tables of mean daily stage and range in stage will indicate the days for which the mean daily discharge can be found directly from the mean daily gage height and those for which more frequent intervals are necessary.

Refinement of gage heights.—The degree of refinement necessary to give a sufficiently accurate determination of discharge will vary inversely with the stage and is determined by the sensitiveness (p. 46) of the station as disclosed by a study of the discharge rating table.

Gages are usually read to hundredths, quarter-tenths, half-tenths, or tenths. The resulting absolute error of observations in individual readings are shown by the following table:

Absolute errors for individual gage readings.

	Maximum error.	Average error.	
	Foot.	*Foot.*	*Fractional parts of Tenths of a foot.*
Readings to hundredths	0.005	0.0025	$\frac{1}{32}$
Readings to quarter-tenths	.012	.006	$\frac{1}{16}$
Readings to half-tenths	.025	.012	$\frac{1}{8}$
Readings to tenths	.05	.025	$\frac{1}{4}$

For staff and chain gages 2 per cent has been selected, more or less arbitrarily, as the limit of allowable average error in a daily discharge due to errors in the mean daily gage height. The table indicates that the maximum error for any one day is twice the average error, so that the maximum error for any one day may be 4 per cent. According to the principles of least squares, for fluctuating stages the average error in the monthly mean discharge resulting from a 2 per cent average error in mean daily discharge is about one-third of 1 per cent.

The refinement to which the mean daily gage-height records must be used—whether to hundredths, half-tenths, or tenths—in order to obtain this limit of accuracy of discharge at any given stage, will depend on the percentage of difference in discharge for such least differences in gage readings at that stage, as shown by the rating table.

In determining this refinement proceed as follows and enter the results in a table of the form given below, in which the Potomac at Point of Rocks, which is read twice daily to tenths, is used as an example.

Limits of accuracy in the use of gage readings.

Station	Present Readings		Minimum Gage Height	Minimum discharge	Error in discharge due to error of .10 ft. in the gage at minimum discharge	Use gage heights to—		
	Per day	To				Hundredths below	Half tenths between	Tenths above
	No.	Foot	Feet	Sec. ft.	Per cent	Feet	Feet	Feet
Potomac River, Point of Rocks, Md.	1	0.1	0.50	900	21	1.0	1.0–2.0	2.0

Enter in column 1, the name of station; in column 2, the number of readings per day; in column 3, smallest subdivision used in reading gage; in column 4, the minimum known gage height; in column 5, minimum discharge as taken from the discharge rating table or curve; column 6, the percentage of error in the minimum discharge due to an error of .10 of a foot in gage height. The discharge rating table (p. 108) shows that the minimum discharge is 900 second-feet and occurs at gage height .50 foot. The difference per tenth between gage heights .50 and .60 is 190 second feet, or 21.1 per cent of the minimum discharge.

The limits of stage between which it is necessary to use mean daily gage heights to hundredths, half-tenths, and tenths, respectively, in order not to introduce an average error of over 2 per cent in the daily discharge are shown in columns 7, 8 and 9 and are determined by trial by testing values from the discharge rating table (p. 108) at selected half-foot intervals as follows:

(a) Testing at the 2-foot gage height for gage records to tenths. The difference between the discharges at 2.00 feet and 2.10 feet is 360 second-feet. The average error of a mean daily record to tenths is one fourth tenth (see table, p. 105). Therefore at gage height 2.00 feet the average error for such record, expressed in second-feet, is $\frac{360}{4} = 90$ second-feet, which is 1.8 per cent of 5,020 second-feet, the discharge at the 2-foot stage. Therefore, it is not necessary to use gage-height records closer than .10 foot above the 2-foot stage, as above this stage the average error is less than 2 per cent, which is the allowable error.

A continuation of this analysis shows that in order to keep the dis-

charge error resulting from lack of refinement in gage readings, below 2 per cent, the gage at Point of Rocks should have been used to hundredths below the 1-foot stage, to half-tenths between 1.0 feet and 2.0 feet, and to tenths above 2.0 feet, instead of to tenths for all stages, as shown in the table of daily gage heights, page 108.

For automatic gage records the same procedure is followed except that the allowable error should be 1 per cent.

For stations with shifting channels the methods of analysis above described can be used only in a general way.

In practice the limits of use of gage heights can be readily determined by the following rules:

Find the stage at which the difference in discharge per tenth is 8 per cent of the discharge at that stage. Gage heights above this stage should be used to tenths.

Find the stage at which the difference in discharge per tenth is 16 per cent of the discharge at that stage. Gage heights below this stage should be used to hundredths.

Gage heights between the first and second stages should be used to half-tenths.

The following tables and figs. 24 and 26 illustrate the method of determining daily discharge of streams with permanent beds:

Discharge measurements of Potomac River at Point of Rocks, Md., in 1902–7.

Date.	Hydrographer.	Area of section.	Mean velocity.	Gage height.	Discharge
1902		*Sq. ft.*	*Ft. per sec.*	*Feet.*	*Sec.-ft.*
June 22	Newell and Paul	2,897	1.01	1.25	2,921
Sept. 2	E. G. Paul	2,356	.73	.87	1,717
1903					
Mar. 12	E. C. Murphy	6,600	2.86	4.84	18,880
Apr. 17	Hoyt and Paul	17,250	5.01	13.70	86,420
Apr. 17	Hoyt and Stokes	16,500	4.88	13.10	80,520
Apr. 18	Hoyt and Stokes	12,180	4.44	9.60	54,080
Sept. 14	Paul and Sawyer	2,950	1.28	1.50	3,770
Nov. 9	W. C. Sawyer	2,590	.83	1.12	2,140
1904					
July 11	Hoyt and Grover	5,500	2.50	3.87	13,750
1905					
Mar. 13	Tillinghast and Comstock	8,600	3.33	6.56	28,640
June 20	Grover and Lyman	2,727	1.10	1.29	2,997
Oct. 30	G. F. Harley	3,532	1.38	2.05	4,889
Nov. 9	G. F. Harley	2,703	.94	1.20	2,531
Nov. 9	Harley and Stewart	2,703	.91	1.20	2,467
1906					
May 30	R. Follansbee	3,351	1.16	1.70	3,892
Dec. 7	R. H. Bolster	3,180	1.40	1.76	4,450
1907					
Mar. 15	R. H. Bolster	21,460	5.31	16.95	114,000

Rating table for Potomac River at Point of Rocks, Md., from April 1, 1902, to December 31, 1906.

Gage height.	Dis-charge.	Differ-ence.	Gage height.	Dis-charge.	Differ-ence.	Gage height.	Dis-charge.	Differ-ence.
Feet.	*Sec.-ft.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Sec.-ft.*
0.50	900	....	2.40	6,520	390	4.60	17,430	1,160
.60	1,090	190	.50	6,920	400	.80	18,610	1,180
.70	1,295	205	.60	7,330	410	5.00	19,820	1,210
.80	1,515	220	.70	7,750	420	.20	21,060	1,240
.90	1,750	235	.80	8,180	430	.40	22,300	1,240
1.00	2,000	250	.90	8,620	440	.60	23,560	1,260
.10	2,260	260	3.00	9,070	450	.80	24,840	1,280
.20	2,530	270	.10	9,530	460	6.00	26,140	1 300
.30	2,810	280	.20	10,000	470	.20	27,460	1,320
.40	3,100	290	.30	10,480	480	.40	28,780	1,320
.50	3,400	300	.40	10,970	490	.60	30,100	1,320
.60	3,700	300	.50	11,470	500	.80	31,460	1,360
.70	4,010	310	.60	11,980	510	7.00	32,820	1,360
.80	4,330	320	.70	12,490	510	.50	36,340	3,520
.90	4,670	340	.80	13,010	520	8.00	39,980	3,640
2.00	5,020	350	.90	13,530	520	.50	43,740	3,760
.10	5,380	360	4.00	14,070	540	9.00	47,600	3,860
.20	5,750	370	.20	15,150	1,080	.50	51,560	3,960
.30	6,130	380	.40	16,270	1,120	10.00	55,600	4,040

NOTE: The above table is applicable only for open-channel conditions. It is based on discharge measurements made during 1902 to 1907. It is well defined between gage heights 1.0 feet and 17.0 feet. Above gage height 10 feet the rating curve is a tangent, the difference being 830 per tenth.

Daily gage heights and discharges of Potomac River at Point of Rocks, Md., for July to December, 1904.

Day.	July.		August.		September.		October.		November.		December.	
	Gage height.	Dis-charge.	Gage height.	Dis-charge.	Gage height.	Dis-charge.	Gage height.	Dis-charge.	Gage height.	Dis-charge.	Gage height.	Dis-charge.
	Feet.	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*
1	1.4	3,100	1.4	3,100	0.9	1,750	0.6	1,090	0.6	1,090	0.8	1,515
2	1.3	2,810	1.3	2,810	.8	1,515	.6	1,090	.6	1,090	.8	1,515
3	1.3	2,810	1.2	2,530	.8	1,515	.6	1,090	.7	1,295	.8	1,515
4	1.3	2,810	1.2	2,530	.8	1,515	.6	1,090	.7	1,295	.8	1,515
5	1.2	2,530	1.2	2,530	.9	1,750	.6	1,090	.7	1,295	.8	1,515
6	1.5	3,400	1.2	2,530	1.0	2,000	.6	1,090	.7	1,295	.8	1,515
7	1.5	3,400	1.3	2,810	.9	1,750	.6	1,090	.7	1,295	.8	1,515
8	1.6	3,700	1.4	3,100	.8	1,515	.6	1,090	.7	1,295	.8	1,515
9	1.6	3,700	1.5	3,400	.7	1,295	.5	900	.7	1,295	.8	1,515
10	1.7	4,010	1.4	3,100	.7	1,295	.5	900	.7	1,295	.9	1,750
11	2.9	8,620	1.3	2,810	.7	1,295	.5	900	.7	1 295	.9	1,750
12	2.6	7,330	1.3	2,810	.8	1,515	.6	1,090	.7	1,295	.9	1,750
13	3.4	10,970	1.2	2,530	1.0	2,000	.7	1,295	.7	1 295	.9	1,750
14	3.1	9 530	1.2	2,530	.9	1,750	.7	1,295	.8	1,515	.9	1,750
15	3.0	9,070	1.1	2,260	.9	1,750	.6	1,090	.8	1 515	.9	1,750
16	2.8	8,180	1.1	2,260	1.0	2,000	.6	1,090	.8	1,515	.9	1,750
17	2.4	6,520	1.1	2,260	1.0	2,000	.5	900	.7	1,295	.9	1,750
18	2.0	5,020	1.0	2,000	.9	1 750	.5	900	.8	1,515	.9	1,750
19	1.8	4,330	1.0	2,000	.8	1,515	.5	900	.8	1,515	.9	1,750
20	1.6	3,700	1.0	2,000	.7	1,295	.6	1,090	.8	1,515	1.0	2,000
21	1.4	3,100	1.0	2,000	.8	1,515	1.0	2,000	.7	1,295	1.0	2,000
22	1.4	3,100	.9	1,750	1.0	2,000	.9	1,750	.7	1,295	1.0	2,000
23	1.3	2,810	.9	1,750	.9	1,750	.8	1,515	.7	1,295	1.0	2,000
24	1.3	2,810	1.0	2,000	.8	1,515	.7	1,295	.7	1,295	1.0	2,000
25	1.3	2,810	1.0	2,000	.8	1,515	.7	1,295	.7	1,295	1.1	2,260
26	1.4	3,100	1.2	2,530	.8	1,515	.7	1,295	.7	1,295	1.4	3,100
27	1.4	3,100	1.1	2,260	.7	1,295	.7	1,295	.7	1,295	1.5	3,400
28	1.4	3,100	1.1	2,260	.7	1,295	.7	1,295	.7	1,295	1.8	4,330
29	1.5	3,400	1.0	2,000	.7	1,295	.6	1,090	.8	1,515	1.8	4,330
30	1.5	3,400	1.0	2,000	.7	1,295	.6	1,090	.8	1,515	1.9	4,670
31	1.5	3 400	.9	1,750	...	....	.6	1,090	....	....	2.0	5,020
Total.		139,670		74,200		47,760		36,080		40,200		68,245
Mean.		4,505		2,394		1,592		1,164		1,340		2.201

Monthly discharge of Potomac River at Point of Rocks, Md., for **1904.**

Month.	Discharge in second-feet.				Run-off.	
	Maximum.	Minimum.	Mean.	Second-feet per sq. mi.	Depth in inches.	Acre-feet.
July	10,970	2,530	4,505	.467	.538	277,300
August	3,400	1,750	2,394	.248	.286	147,200
September	2,000	1,295	1,592	.165	.184	94,730
October	2,000	900	1,164	.121	.140	71,570
November	1,515	1,090	1,340	.139	.155	79,740
December	5,020	1,515	2,201	.228	.263	135,300
The period	10,970	900	2,199	.228	1.556	805,840

GAGING STATIONS WITH CHANGEABLE BEDS.

The determination of the daily discharge of streams with changeable beds is more difficult than of those with permanent beds. The method used varies with the rapidity of the changes. The base data for such determinations are the same as those used for permanent beds, but more frequent discharge measurements are necessary, as otherwise the results obtained are only roughly approximate.

PERIODICALLY CHANGING BEDS.

For stations with beds which shift slowly or are changed only during floods, station rating curves can be prepared as above described for periods between changes, and satisfactory results can be obtained with two or three measurements a month, provided measurements are taken soon after such changes take place.

CONSTANTLY CHANGING BEDS.

For streams with continually shifting beds, as the Colorado and Rio Grande, discharge measurements should be made every two or three days and the discharge for the intervening days estimated by interpolation, modified by the gage heights for these days. There are two methods of making these interpolations, the Stout and the Bolster methods, known by the names of their inventors.

Stout method. In the Stout method an approximate station rating curve and rating table are prepared from the discharge measurements and applied to modified or so-called corrected daily gage heights. The gage heights are corrected by means of a curve (fig. 27) determined by plotting as ordinates the differences between the actual gage heights at the time of the various discharge measurements and the gage height

corresponding on the approximate curve to the respective measured discharges, and as abscissas the corresponding days of the months. Through these points an irregular curve is drawn, from which can be found the correction for days other than those on which measurements were made. The correction is positive if the discharge is greater than that given by the station rating curve, negative if less. Each daily gage height is then corrected by the amount indicated on the correction curve, and the discharge corresponding thereto is taken from the approximate rating table.

Bolster method.—In the Bolster method the discharge measurements for the entire year are first plotted with discharges as abscissas and gage heights as ordinates. The points so plotted are considered chronologically and, even though scattered, will usually locate one or more fairly well-defined curves, called standard curves (fig. 27). In general the number and position of these standard curves is determined by the radical changes in the stream bed due to floods.

When beds change very rapidly it is necessary to change the position of the rating curve from day to day, making practically a new curve daily. This daily curve is of the same form as the standard curve and is parallel to it with respect to ordinates. For a day when a measurement is made the rating curve passes through such plotted measurement. In order to locate a rating curve for other days a line connecting consecutive measurements is drawn and divided into as many equal parts as there are days intervening between the measurements, on the assumption that the change in conditions of flow between any two consecutive measurements is uniform from day to day. The daily rating curve will then pass through these points of division, and the discharge is read directly from these curves by applying to them the observed daily gage heights.

In order to facilitate the use of the method and to make it as rapid in application as the common method for permanent stations the standard curve or curves, together with a vertical line of reference, should be transferred from the original station sheet to tracing cloth, which can be readily shifted vertically to any desired position by always keeping the two reference lines coincident with each other.

In applying and modifying this method judgment must be used for long time intervals of no measurements, or for radical changes in the stream bed caused by sudden floods. The tables on pages 112–113 and figure 27 illustrate the Bolster and the Stout methods of obtaining daily discharge.

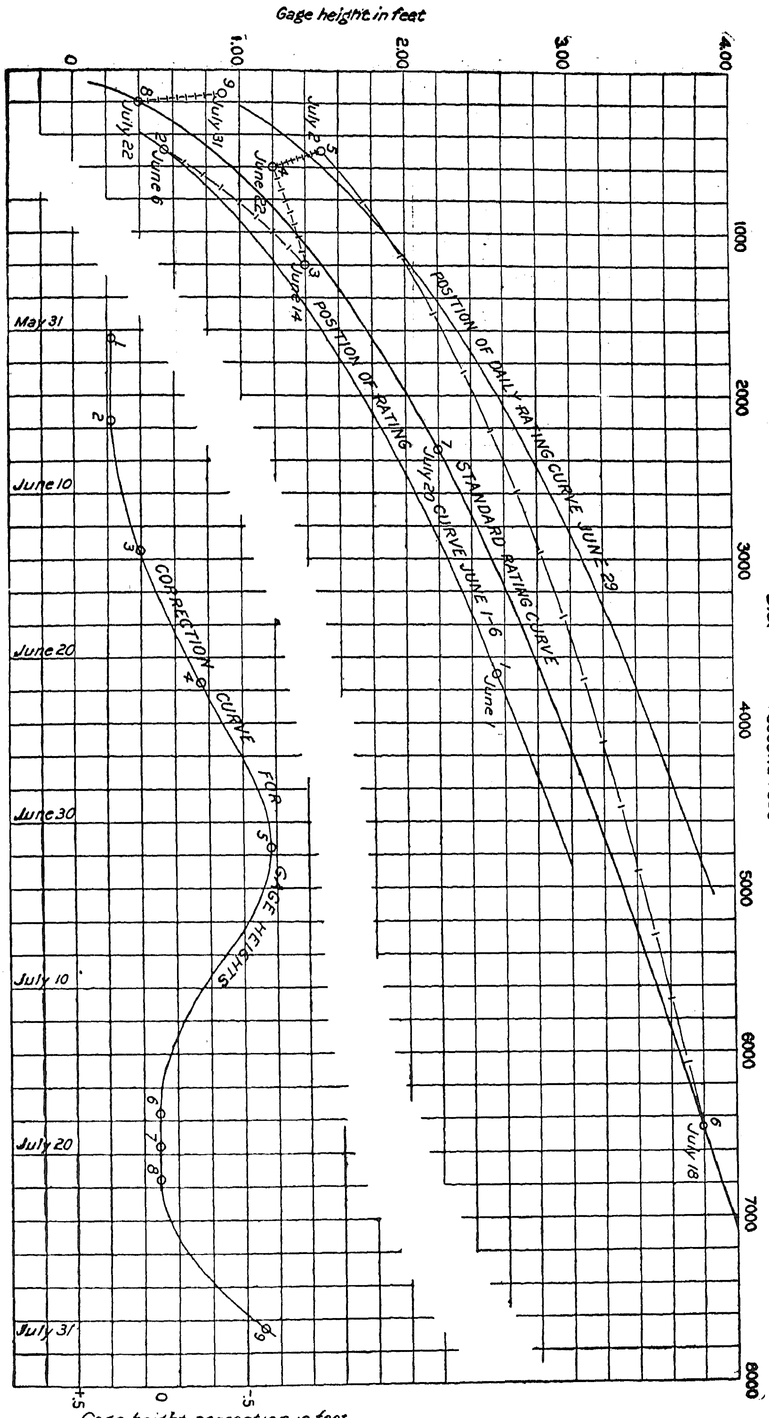


FIG. 27—Typical Curves Illustrating the Stout and the Bolster Methods of Computing Daily Flow.

List of measurements to illustrate the Stout and the Bolster methods of determining daily discharge.

No.	Date.	Gage height.	Discharge.
		Feet.	*Sec.-ft.*
1	June 1	2.55	3700
2	6	.55	500
3	14	1.4	1200
4	22	1.2	600
5	July 2	1.5	500
6	18	3.8	6460
7	20	2.2	2330
8	22	.4	200
9	31	.9	150

Daily gage heights and discharges to illustrate the Stout and the Bolster methods of determining daily discharge.

Day.	June.				July.			
	Observed gage heights.	Discharge, Bolster method.	Corrected gage heights.	Discharge, Stout method.	Observed gage heights.	Discharge, Bolster method.	Corrected gage heights.	Discharge, Stout method
	Feet.	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*
1	3.0	4870	3.3	4970	1.5	530	.85	520
2	2.5	3580	2.8	3650	1.5	500	.85	520
3	2.0	2490	2.3	2530	1.5	570	.85	520
4	1.5	1610	1.8	1640	1.6	730	.95	605
5	1.0	930	1.3	960	1.6	780	1.0	650
6	.4	390	.65	365	1.8	1050	1.25	905
7	.6	510	.85	520	1.7	970	1.2	850
8	.6	490	.85	520	1.7	1020	1.25	905
9	.7	550	.95	605	1.9	1320	1.55	1280
10	.8	620	1.0	650	2.0	1510	1.7	1490
11	1.0	800	1.2	850	2.2	1890	2.0	1970
12	1.5	1400	1.65	1420	2.6	2720	2.45	2850
13	1.4	1240	1.55	1280	3.0	3700	2.9	3900
14	1.4	1200	1.5	1210	3.6	5440	3.5	5550
15	1.3	1020	1.35	1020	3.7	5850	3.65	6000
16	1.1	760	1.15	800	3.5	5370	3.5	5550
17	1.0	620	1.0	650	3.2	4610	3.2	4690
18	1.0	580	.95	605	3.7	6150	3.7	6150
19	1.1	640	1.0	650	4.0	7080	4.0	7080
20	1.2	690	1.05	700	3.0	4160	3.0	4160
21	1.2	650	1.0	650	1.8	1640	1.8	1640
22	1.3	690	1.05	700	.8	480	.8	480
23	1.5	850	1.2	850	.3	120	.3	150
24	1.8	1170	1.45	1140	.3	100	.25	130
25	1.7	990	1.3	960	.4	110	.35	175
26	1.6	830	1.15	800	.4	90	.3	150
27	1.6	780	1.1	750	.6	130	.4	200
28	1.6	740	1.05	700	.7	150	.45	230
29	1.6	700	1.0	650	.8	170	.45	230
30	1.6	660	.95	605	.9	180	.4	200
31	....	....	....	....	1.0	200	.4	200
Totals.		33050		33400		59320		59930
Means.		1102		1113		1914		1933

Rating table to illustrate the Stout method of determining daily discharge.

Gage height.	Dis-charge.	Gage height.	Dis-charge.	Gage height.	Dis-charge.	Gage height.	Dis-charge.	Gage height.	Dis-charge.
Feet.	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*
0.00	60	1.00	650	2.00	1970	3.00	4160	4.00	7080
.10	80	.10	750	.10	2150	.10	4420	......	......
.20	110	.20	850	.20	2330	.20	4690	......	......
.30	150	.30	960	.30	2530	.30	4970	......	......
.40	200	.40	1080	.40	2740	.40	5260	......	......
.50	260	.50	1210	.50	2960	.50	5550	......	......
.60	330	.60	1350	.60	3180	.60	5850	......	......
.70	400	.70	1490	.70	3410	.70	6150	......	......
.80	480	.80	1640	.80	3650	.80	6460	......	......
.90	560	.90	1800	.90	3900	.90	6770	......	......

ICE-COVERED STREAMS.[a]

Ice occurs in rivers in three forms—surface ice, anchor ice and frazil. Surface ice may occur as a complete cover, supported by the water, or bridged from bank to bank, free from or partly supported by the water, as in ice jams due to piling up of ice, or as alternate layers of ice and water. Anchor ice may be attached to the bed of the river where it has been formed, or it may be floating in suspension. Frazil usually occurs floating in suspension.

The presence of ice in a stream in any form may destroy the open-water relation of discharge to stage by causing backwater and thus increasing the stage for a given discharge. Therefore discharge measurements of a stream in which ice is present will always plot either at the left of or on the open-water curve. Under no circumstances will they plot to the right provided the measurement represents the correct flow.

It is not necessary that ice be present at the measuring section or at the normal control section to destroy the relation of stage to discharge. The existence of ice far below the control section may establish a temporary control which will affect the station rating.

The data necessary for computing flow during periods of ice are—

1. Measurements of discharge made during the period.

2. Records of stage read to the surface of the water in a hole cut through the ice.

3. Records of temperature and precipitation.

4. Full notes in regard to the ice.

The complex manner in which ice may form and the varying conditions presented by streams preclude the formulation of any method that can be universally employed to determine winter flow. In general the following methods may be used:

[a] See Water-Supply Paper No. 337, U. S. Geol. Survey.

1. The readings of gage heights to the water surface may be directly applied to the open-water rating curve.

2. The observed gage heights may be applied directly to a special rating curve based on winter discharge measurements and gage heights to water surface.

3. Discharge measurements may be used in connection with gage heights and with data showing climatic conditions and the occurrence of ice. This method may be applied either by the eye or graphically, as will be explained, for determining corrections to the gage heights necessary for making the open-channel rating curve applicable.

FIRST METHOD.

The use of the first method to determine the daily discharge of a frozen stream—the application of water-surface gage readings to the open-water rating curve—is advisable only when the stream is open at the control section and no backwater exists at the gage.

If the control section is entirely free from ice the relation between slope, stage, and discharge will not be appreciably changed even by complete ice cover between the control section and the gage.

In using this method the engineer should closely inspect the gage-height records and compare them with temperature records to detect the presence of backwater. If discharge measurements made in several winters have shown that ice rarely occurs at the control section and that the regular open-water curve is applicable fewer measurements are needed with this method than with any other. An open-control section, however, with ice above, implies as a rule that the control section is at rapids at which anchor ice is likely to form. In order to detect the presence of anchor ice during extremely cold periods the gage should be read twice a day. Readings higher in the morning than in the afternoon indicate the presence of anchor ice, and care must be taken to read the gage soon after the point of maximum daily temperature, when the control section is likely to be clear.

SECOND METHOD.

The conditions favorable to the use of the second method—in which observed gage heights are applied directly to a special rating curve based on winter discharge measurements and gage heights to the water surface—are most likely to be found on the larger rivers, where the slope and cross-section may be fairly uniform for long stretches. In general, however, this method should be used with great care and the period of

applicability of the curve should be closely determined by discharge measurements.

On a large stream that freezes uniformly and that varies little in flow all discharge measurements made when the ice has reached its permanent condition may plot on a curve, but it is evident that this curve will not correctly represent the relation of stage to discharge in the period of transition from open water to solid ice, and vice versa. It will also not represent conditions if between the times of measurement the character of the ice cover has changed greatly as a result of changes in temperature.

If backwater is caused by a combination of anchor and surface ice, discharge measurements made at certain times might give a smooth curve that would in reality not apply except on the days of measurement.

This method should be used only when many discharge measurements have been found to plot on a smooth curve and when conditions of temperature and ice are stable for long periods.

THIRD METHOD.

Eye method.—The method most commonly employed at the present time to determine the flow of streams either partly or completely covered with ice is that which utilizes the discharge measurements and data regarding climatic conditions and the occurrence of ice in connection with observed gage heights by means of eye-inspection of records of temperature, precipitation, and gage heights, estimating the daily discharge for the period between times of measurement, and adjusting the determinations by comparing with results obtained at near-by stations. The monthly mean of such determinations is also compared with monthly means at adjacent stations in order that any large error may be detected.

The accuracy of this method depends largely on the uniformity of stream flow between times of measurements, the number of measurements, and the engineer's knowledge of general conditions. Care must be taken that the discharge, as estimated, is not greater than would be given by the application of gage heights to the open-water rating curve. In general this method should give more accurate results than the second method, because it considers the time and temperature factors.

The method will give good results at stations in localities where temperatures are fairly constant over long periods of time and where the flow is affected by surface rather than by anchor ice. Under such conditions fewer measurements and gage readings are required than at

stations situated where climatic conditions are irregular. Temperature records from the nearest Weather Bureau station are generally sufficient.

The disadvantage of this method is that it is impossible to check its results, as no record is left of steps employed.

Graphic method.[a]—As ice causes backwater it is only necessary to determine the magnitude of the backwater effect in order to find the true flow at a given stage. Since the formation of ice is due entirely to temperature, the amount of backwater will in general vary with temperature.

The magnitude of the backwater effect at any given time can be determined by measurements of discharge. If such measurements are made at stated intervals during the winter the backwater effect between times of measurement can be determined by constructing a curve of backwater. In constructing this curve (see fig. 28) proceed as follows:

1. Plot the observed daily gage heights.
2. Plot the mean daily temperatures.
3. Plot the daily precipitation.
4. Plot on the curve of observed daily gage heights the gage heights corresponding to the measured discharges as determined from the open-water rating table. The differences between these gage heights and the observed gage heights measure the backwater effects for the days in question.
5. Plot the backwater effects as determined under 4 and through these plotted points construct a backwater curve, following the same general shape as the inverted temperature curve, taking into account the daily precipitation (if rain), ice jams, and other unusual conditions that may affect the records of stage.
6. From the backwater curve construct the curve of corrected gage heights, from which the true discharge can be obtained by applying the open-water rating table.

Aside from giving more accurate results, this method has an advantage over any other method in furnishing a complete record of all the steps taken, thus making it possible for a second person to review or check the estimates. The accuracy of the results obtainable by this method will depend on the frequency of the discharge measurements. When winter conditions are comparatively constant, fewer measurements will be required, as the principal uncertainties occur during transition periods from cold to warm weather.

A special form, see fig. 29, is desirable for use in computing winter flow.

[a] The graphic method was first proposed by W. G. Hoyt, Assoc. M. Am. Soc. C. E., Eng. News Vol. 69, pp. 725–727, Apr. 10, 1913.

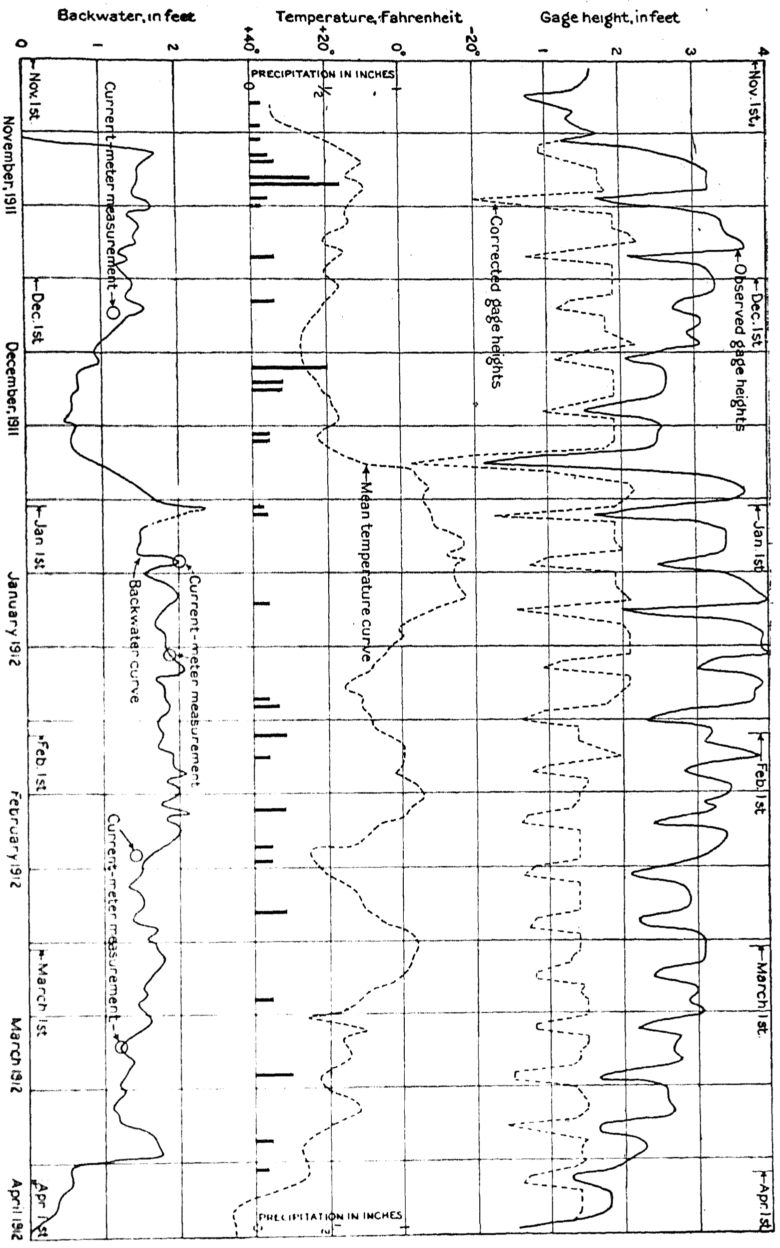


Fig. 28.—Gage height, backwater, temperature and precipitation curves. Rainy River. International Falls, Minn.

This method follows somewhat the Stout method (pp. 109–110) of determining the flow in shifting channels in using a single rating curve and correcting the observed gage heights to apply to the curve. In the

COMPUTATION OF FLOW DURING WINTER PERIOD OF.................RIVER

AT
NEAR.........................Month of...................191

DAY	Observed											Computed	
	Water height at gage					Temperature			Weather	Ice at control	Results of discharge measurements	Effective gage height	Estimated discharge, second-feet
	Morning		Afternoon		Mean height								
	Time	Height	Time	Height		High	Low	Mean					
1													
2													
3													
4													
5													
6													
7													
8													
9													
10													
11													
24													
25													
26													
27													
28													
29													
30													
31													

Mean discharge	Sec. ft.
..........to..........	
..........to..........	
..........to..........	
..........Maximum..........	
..........Minimum..........	

Total.................

Mean.................

Computed by.................

Checked by.................

FIG. 29.—Proposed computation sheet.

Stout method the correction is based almost entirely on discharge measurements; in this ice method time and temperature are factors, as well as discharge measurements.

Application of graphic method.—Many streams in the northern and western States are not completely covered with ice throughout the winter months. At many places the ice on a stream may extend but a short distance from the banks, leaving an open channel in the center; at others the current may be so swift that the channel remains entirely open throughout the winter. Such conditions are common in hilly or mountainous regions, where stream gradients change rapidly, and may exist in the vicinity of gaging stations to such an extent as to make it impossible to estimate discharge by methods that are applied to streams

that are completely ice covered. For example, the channel may be open at the gage or at the control below, but the stage may be materially affected by the presence of anchor ice; or, if complete ice cover exists below the natural control sections, ice jams may form, creating a temporary ice control which produces backwater at the gage. If the stream is frozen along the banks at the gage and at the control below, but is open in the center, the gage readings are affected by ice jams at the control and by changes in the conditions along the banks.

If a stream is deep and sluggish and under complete ice cover for a considerable distance above a gage located at a riffle, the gage readings may not be affected by ice because the water coming from beneath the ice is at a temperature sufficiently high to prevent the formation of ice at the control section.

The extent of these effects depends on the natural condition of the channel and on the general condition of the ice, the latter depending largely on temperature. Each station where such conditions exist, singly or in combination, presents a problem that can be solved only after a careful study of all the factors.

At a station located in a comparatively sluggish stretch of a stream where the channel is partly open throughout the winter and the control is a riffle the gage heights will be materially affected by anchor ice, which will form at temperatures of zero and below. The amount of anchor ice will depend on the roughness of the channel and the velocity of the current and will tend to increase as the temperature falls and decrease as the temperature rises, disappearing entirely as the temperature approaches 32°. As the variation in daily temperature may have an important effect on the amount of anchor ice formed, and therefore on the fluctuations in stage at the station, a record of the maximum and minimum temperatures is necessary to arrive at a proper interpretation of variation in the gage heights caused by anchor ice. It is also necessary to have a more complete record of the gage heights at stations of this character than at stations where complete ice cover is formed. At least two readings should be made each day as nearly as possible at the time of maximum and minimum temperatures. Careful observations should be made to determine the maximum temperature at which anchor ice forms and at which it may be expected to disappear, and the relation between the increase of the anchor ice and the decrease in temperature. On days when the temperature rises above this maximum, if other conditions are normal, it may be assumed that no anchor ice exists and that the gage heights at or immediately following the time of maximum temperature are reliable. With this information

UNITS OF DISCHARGE.

The second-foot.—The standard term for expressing discharge is the second-foot. As the second-foot is a unit of rate of flow and is in itself indefinite as regards duration, it must be used in connection with some unit of time, as a day, week, month, or year, in which event the term means that for each second during the period of time selected the flow averaged so many cubic feet per second. This has generally been considered the fundamental unit of discharge and from it other terms of discharge and run-off are usually computed, the conversion to other units being a simple arithmetical process, usually made by means of tables specially prepared for the purpose.

Gallons per minute.—Gallons per minute is generally used in connection with pumping and city water supply.

Miner's inch.—The miner's inch is usually defined as the quantity of water which passes through an orifice one inch square under a fixed head, which varies locally. It has been commonly used by miners and irrigators throughout the West, and is defined by statute in many States. Owing to the confusion caused in measuring the miner's inch, and to the fact that as formerly defined by size of orifice and head it was not exact, it is now defined as a certain part of a second-foot, usually $\frac{1}{50}$ or $\frac{1}{40}$.

Second-feet per square mile.—Second-feet per square mile is the average number of cubic feet of water flowing per second from each square mile of area drained, on the assumption that the discharge is distributed uniformly both as regards time and area. It is found by dividing the mean discharge in second-feet by the drainage area.

UNITS OF RUN-OFF

Run-off in inches.—Run-off in inches is the depth to which a plane surface equal in extent to the drainage area would be covered if all the water flowing from it in a given period were conserved and uniformly distributed thereon. It is used for comparing run-off with rainfall, which is usually expressed in depth in inches.

Acre-foot.—An acre-foot is equivalent to 43,560 cubic feet, and is the quantity required to cover an acre to the depth of one foot. It is commonly used in connection with storage. There is a convenient relation between the second-foot and the acre-foot. One second-foot flowing for twenty-four hours will deliver 86,400 cubic feet, or approximately 2 acre-feet. One acre-foot equals 325,851 gallons, or a million gallons is somewhat more than 3 acre-feet.

On pages 201-203 are given conversion tables for the various units of discharge and run-off.

ACCURACY OF STREAM-FLOW DATA.[a]

DEGREE OF ACCURACY REQUIRED.

The laws relating to many natural phenomena have been reduced to an exact science; those for many more are largely empirical and are based on experiments and assumptions which only approximate the truth. In the empirical class are included the laws of the science of hydrology and especially of that branch of hydrology which relates to the flow of water in open channels. It is possible, nevertheless, by carefully considering the various factors, to reduce the incidental errors so that resulting records will be sufficiently accurate for the purposes for which stream-flow data are required.

In most problems two degrees of accuracy must be considered, first, that which is practicable or possible to obtain, and second, that which is desirable or necessary. The obtainable accuracy of stream-flow data depends largely on the amount of money available for their collection. The desirable accuracy will depend on the proposed use of the data.

Variations in maximum, minimum and mean discharges in second-feet of certain rivers.

	Connecticut River, Sunderland, Mass., 1904-1912.	Susquehanna River, Harrisburg, Pa., 1891-1912.	Coosa River, Riverside, Ala., 1897-1912.	Allegheny River, Kittanning, Pa., 1905-1913.	Rio Grande, San Marcial, N.Mex., 1897-1913.	Colorado River, Yuma, Ariz., 1903-1912.	Merced River, Merced Falls, Cal., 1901-1911.	Sacramento River, Red Bluff, Cal., 1902-1912.	Columbia River, The Dalles, Oreg., 1879-1913.
Drainage area	7,700	24,000	7,060	9,010		242,000	1,090	10,400	237,000
Yearly mean	14,000	37,000	12,000	16,900	1,590	24,400	1,820	15,600	213,000
Smallest yearly mean . .	11,900	29,300	4,940	13,300	207	13,900	687	9,340	130,000
Largest yearly mean . . .	16,500	54,500	16,400	20,500	3,340	35,800	2,920	21,600	336,000
Mean yearly maximum .	79,900	270,000	57,500	155,000	12,100	95,500	15,000	138,000	650,000
Largest yearly maximum	103,000	544,000	75,800	269,000	33,000	150,000	37,200	254,000	1,160,000
Smallest yearly maximum	62,100	172,000	20,900	62,100	4,070	51,200	3,740	55,000	302,000
Mean yearly minimum . .	1,880	4,650	2,540	1,170	40	4,300	50	4,740	65,500
Largest yearly minimum .	3,950	10,200	4,380	2,410	685	6,800	80	5,470	103,000
Smallest yearly minimum	1,110	2,570	1,220	570	0	2,690	0	4,200	41,900

Stream-flow records have three principal uses: First, in predicting future flow, generally in connection with the design of hydraulic works; second, in the immediate operation of hydraulic works, and third, in studying conditions of past flow, usually in connection with the adjust-

[a] See U. S. Geol. Survey Water-Supply Paper No. 95.

ment of water rights. For the second and third uses, data as accurate as can be collected may be needed. In considering the first use, however, it should be remembered that both the total flow of a stream and its regimen change from year to year, and that the conditions existing at any particular time may never recur. For this use, therefore, reasonably accurate records that extend over a considerable period are much more valuable for predicting flow than extremely accurate data covering a short period. The preceding table of variations in discharge of a few typical streams shows the wide range of possibilities that must be considered in designing hydraulic works for various purposes.

CONDITIONS AFFECTING ACCURACY OF DAILY DISCHARGE RECORDS.

The obtainable accuracy of records of daily discharge of a stream depends on—

1. Permanence of the relation of discharge to stage.
2. Probable error of the discharge rating curve.
3. Refinement of gage readings.
4. Frequency of gage readings.
5. Methods of applying the daily gage heights to the rating table to obtain daily discharge.

The conditions affecting accuracy may introduce errors which may be (*a*) consistently compensating, (*b*) consistently cumulative, or (*c*) alternately compensating and cumulative. Therefore care must be taken to determine the way in which the incidental errors affect the results.

The study of the accuracy of records to be collected at any stream should begin with the selection of the site and continue through the establishment, maintenance, and operation of the station and the interpretation of the data, and not be left for determination after the estimates have been made. In other words, records should be collected with view to a desired degree of accuracy instead of leaving their accuracy to be determined after the field data are collected and estimates made.

Permanence of the relation of discharge to stage.—The permanence of the relation of discharge to stage as determined by the control is a fundamental factor, as stated on pages 45 and 46, entering into the collection of records of daily discharge of a stream. The general character of the control is determined by inspection. Its effectiveness, however, can be finally determined only by plotting the results of discharge measurements. If such plotting does not define a smooth curve, the inconsistency is due either to the instability of the control or to disturbing influences. If conditions at the control are unstable, the

accuracy of the record will depend on the number of the discharge measurements and their distribution as to time and stage.

Errors due to lack of permanence of the relation of discharge to stage may be either compensating or cumulative, according to the physical conditions affecting the nature and stability of the control.

Probable error of the discharge rating curve.—The probable error of the discharge rating curve depends on the accuracy of the discharge measurements and the permanence of the control. If the relation of discharge to stage were permanent and truly defined by the rating curve, and the discharge measurements were absolutely accurate, a series of measurements for a station would plot on a smooth curve. Unfortunately, such ideal conditions do not exist; therefore, a series of measurements for a station will plot somewhat discordantly and the rating curve will be drawn among them in such a way as to represent average conditions. For permanent conditions of control with a good measuring section the variations of individual measurements from the mean curve should be comparatively small and as likely to be plus as minus. The probable error of a rating curve may be computed by the method of least squares and will be a factor in determining the probable error of the estimates of daily discharge.

Errors in daily discharge resulting from errors in the position of the rating curve will be cumulative for any stage but may be partly compensating if the curve used lies first on one side and then on the other side of the true curve.

Refinement of gage readings.—Refinement of gage readings (pp. 105–106) affects the accuracy of stream flow data to a degree dependent on the sensitiveness of the station which, in turn, is determined by the control (p. 46). In general the more sensitive the station the more accurate the records that can be collected by ordinary methods and the less refinement necessary in the gage readings.

Errors due to lack of refinement in reading will generally be compensating, but they may be cumulative when fluctuations in stage are small during a considerable period or when due to systematic personal errors of the observer.

Frequency of gage readings.—The frequency of gage readings is an important factor in accuracy of records of streams subject to considerable daily fluctuation in stage. To obtain a gage record of such accuracy that its use with the rating table will give the true mean discharge for the day the number of readings should vary, according to the nature of the fluctuations, from one or two daily to a continuous record obtained by some form of recording gage. The requirements for gage readings

must be determined in advance for the operation of each station. Figure 38 shows daily variations in stage for typical streams.

Errors due to insufficient gage readings may be cumulative or compensating, or alternately one and the other, according to the nature of the fluctuations in stage.

Methods of applying the daily gage heights to the rating table.—The methods of applying daily gage heights to the rating table to obtain mean daily discharge are described on pages 104 and 105. The errors resulting from this operation will in general be cumulative, and their magnitude will vary with the method used in making the computations.

ACCURACY OF MONTHLY OR YEARLY MEANS.

The foregoing discussion of accuracy relates only to daily discharge. For many uses the mean flow for longer periods may be sufficient. The monthly mean is in general use for hydraulic studies. If errors resulting from all causes in the estimates of daily discharge are compensating, the probable error of the mean monthly discharge will be much less than the probable error of the individual daily discharges. A careful analysis of the estimates of daily discharge and monthly means computed from them shows that large errors in the daily estimates may be so compensated that the errors in the monthly means are small.

In this discussion of accuracy it has been assumed that personal or instrumental errors both in field and office are so reduced as to be negligible. In order that this assumption may be true, however, all operations connected with the work must be carefully conducted and instruments must be kept in proper working order.

GRAPHICAL ANALYSIS OF STREAM-FLOW DATA.

In most studies involving stream-flow data, graphical methods of analysis have been found to be serviceable. Common hydrographs, duration curves and summation hydrographs are in common use. The method adopted in any instance will depend largely on the character of the investigation. Following is a partial list of publications illustrating various graphical methods of analysis:

Report on New York water supply, John R. Freeman, 1900.
Water-Supply Paper 198, U. S. Geol. Survey.
Water-Supply Paper 369, U. S. Geol. Survey.
Colorado College Publications, General Series No. 57, 1911.
Engineering News, vol. 70, pp. 496 and 1290; vol. 71, p. 903.

American Civil Engineers Pocket Book, 1st edition.
Water Power Engineering, Daniel W. Mead, 2d edition.
Sixth Annual Report, New York State Water Supply Commission.
Sixth Annual Report, Hydro Electric Power Commission of Ontario.

COMMON HYDROGRAPH.

In order to show for comparative purposes the daily and seasonal distribution of flow of a stream, hydrographs (fig. 30) are prepared by plotting in order of occurrence the discharge for each day during the year and connecting the points so plotted by a curve. These curves are of use not only in studying the variations in flow from year to year, but may also be used in storage problems, where the total quantity of water is an essential factor.

DURATION CURVE.

One of the first steps in investigations for developments that involve the use of water is the determination of the quantity of the available water supply, including the absolute minimum flow, the ordinary

Discharge and horsepower table for Potomac River at Point of Rocks, Md., for 1904.

Discharge in sec.-ft.	Horse-power (80% eff.) per foot fall.	Number days duration between consecutive values of discharge in first column.													Days of deficient discharge.
		Jan.	Feb.	March.	April.	May.	June.	July.	August.	Sept.	Oct.	Nov.	Dec.	Totals.	
900	82	. .	. .	. .	. .	. .	. .	. .	. .	. .	0			0	. . . .
990	90	. .	. .	. .	. .	. .	. .	. .	. .	. .	6	. .		6	6
1,100	100	. .	. .	. .	. .	. .	. .	. .	. .	. .	15	2	. .	17	23
1,320	120	. .	. .	. .	. .	. .	. .	. .	. .	8	7	20	. .	35	58
1,540	140	. .	. .	. .	. .	. .	. .	. .	. .	10	1	8	9	28	86
1,760	160	. .	. .	. .	. .	. .	. .	. .	3	7	1	. .	10	21	107
1,980	180	. .	. .	. .	. .	. .	. .	. .	. .	. .	. .				. . . . .
2,200	200	. .	. .	. .	. .	. .	. .	. .	8	5	1		5	19	126
2,750	250	. .	. .	. .	. .	. .	. .	1	12	. .			1	14	140
3,300	300	. .	. .	. .	. .	. .	1	12	7	. .	. .		1	21	161
3,850	350	17	. .	. .	3	. .	5	8	1	. .			1	35	196
4,400	400	2	. .	. .	3	. .	2	2	. .				2	11	207
4,950	450	. .	. .	. .	2	. .	. .	. .	. .				1	3	210
5,500	500	2	. .	1	4	1	3	1					1	13	223
6,600	600	1		7	5	10	2	1						26	249
7,700	700	. .		4	4	6	4	1						19	268
8,800	800	. .	2	3	1	4	2	2						14	282
9,900	900	2	1	1	4	3	1	2						14	296
11,000	1,000	2	3	2			1	1						9	305
13,200	1,200	1	7	3	3	2	1							17	322
15,400	1,400	1	3	3		2	2							11	333
17,600	1,600		1	3			2							6	339
19,800	1,800		3	1		1								5	344
22,000	2,000	2	1	2			2							7	351
27,500	2,500		4	1		2								7	358
33,000	3,000		3		1		1							5	363
38,500	3,500	1	1				1							3	366
Total days.		31	29	31	30	31	30	31	31	30	31	30	31	366	. . . .

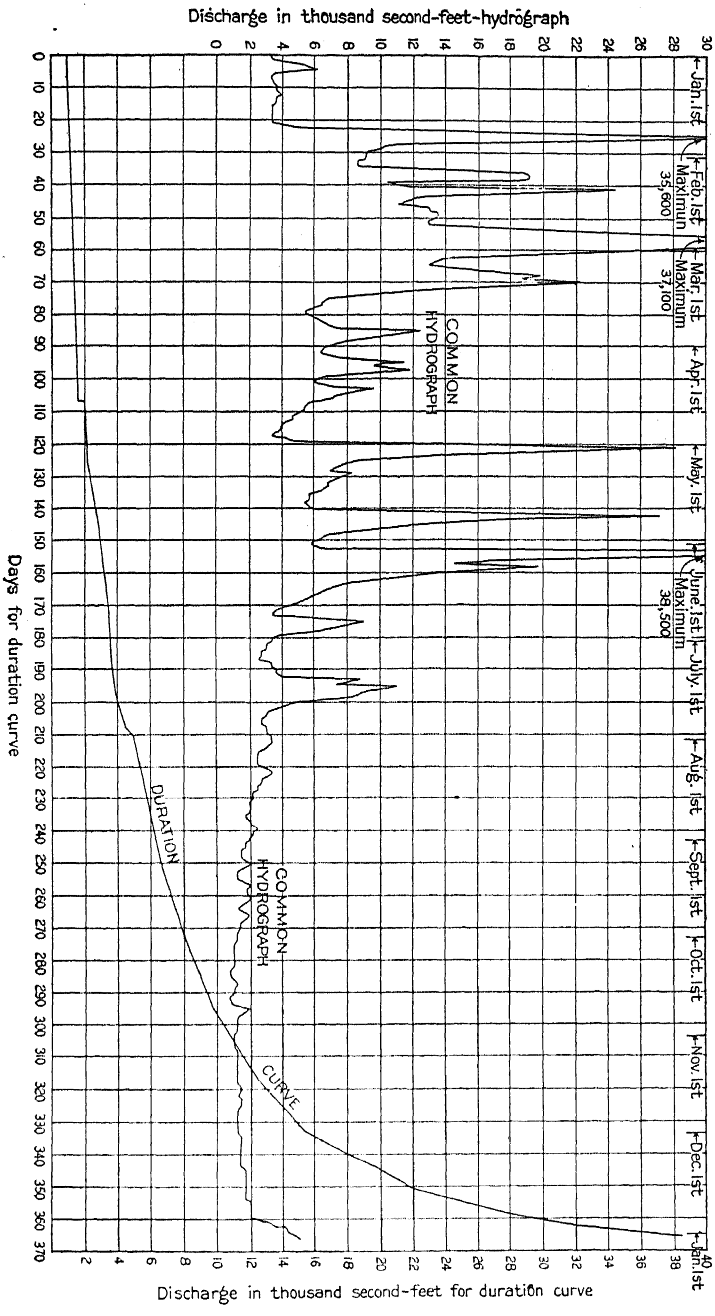


FIG. 30.—Common hydrograph and duration curve for 1904, Potomac River, Point of Rocks, Md.

minimum flow, and the maximum flow. For each the duration as well as the quantity of flow is important. In order that estimates may be reliable several years' records of daily flow should be available for study and comparison.

A knowledge of the duration of the flow of various magnitudes is frequently of value. A yearly table of duration of flow may be constructed by arranging in parallel columns the values of the various daily flows in order of their magnitude and the number of days of the year on which each flow occurs, as shown in the table on page 126. The sum of the numbers in the "Number-of-days" column up to any given flow will be the number of days when the flow is less than that indicated in the "Discharge" column, or the number of days of deficiency.

By plotting the discharge as abscissas and the number of days of deficiency as ordinates, a curve (fig. 30) may be drawn showing the number of days in the year when the discharge is below any given quantity. The horsepower per foot of fall corresponding to the various discharges may also be computed and tabulated.

The duration curve is especially designed for use in studies where no storage is contemplated. If storage is to be utilized the order of occurrence of the flows of various magnitudes is important.

SUMMATION HYDROGRAPH.

Summation hydrographs or mass curves furnish an effective means of making studies of stream-flow data in connection with questions of storage and use of water. The method appears to have been first suggested by W. Rippl.[a]

In the preparation of a summation hydrograph of stream flow accumulated totals of run-off are plotted as ordinates and corresponding times as abscissas. The totals may be expressed in any unit of run-off. The unit of time will commonly be the month, although a longer or shorter period may be used.

On Plate VIII the broken line ACKGEB is a summation hydrograph for the South Branch of Zumbro River, Minn., from January 1, 1910, to April 30, 1912. The application of this hydrograph to the study of certain problems of storage on that river is shown by other lines and diagrams on the plate. In the construction of this hydrograph the monthly discharges for the period are tabulated in second-feet in column two of the following table:

[a] See Proceedings, Institution of Civil Engineers, vol. LXXI, 1883.

Data and computations for summation hydrograph for South Branch of Zumbro River, Minnesota.

Date.	Monthly discharge.	Monthly run-off.	Estimated depth of evaporation.	Evaporation loss from 900 acres.	Net Monthly run-off.	Accumulated run-off.
(*End of month*)	(*Sec.-ft.*)	(*Billion cu.-ft.*)	(*Inches.*)	(*Billion cu.-ft.*)	(*Billion cu.-ft.*)	(*Billion cu.-ft.*)
1910						
January	254	0.680	0.9	0.003	0.677	0.677
February	199	.481	.9	.003	.478	1.155
March	1168	3.129	1.5	.005	3.124	4.279
April	254	.658	2.7	.009	.649	4.928
May	208	.557	3.55	.012	.545	5.473
June	150	.389	3.79	.012	.377	5.850
July	117	.313	4.64	.015	.298	6.148
August	119	.319	3.76	.012	.307	6.455
September	112	.290	3.64	.012	.278	6.733
October	108	.289	2.1	.007	.282	7.015
November	112	.290	2.0	.006	.284	7.299
December	107	.287	1.3	.004	.283	7.582
1911						
January	105	.281	.9	.003	.278	7.860
February	354	.856	.9	.003	.853	8.713
March	171	.458	1.5	.005	.453	9.166
April	142	.368	2.07	.007	.361	9.527
May	174	.466	3.55	.012	.451	9.981
June	118	.306	3.79	.012	.294	10.275
July	81	.217	4.64	.015	.202	10.477
August	245	.656	3.76	.012	.644	11.121
September	97	.251	3.64	.012	.239	11.360
October	1110	2.973	2.1	.007	2.966	14.326
November	271	.702	2.0	.006	.696	15.022
December	438	1.173	1.3	.004	1.169	16.191
1912						
January	165	.442	.9	.003	.439	16.630
February	165	.414	.9	.003	.411	17.041
March	855	2.289	1.5	.005	2.284	19.325
April	930	2.411	2.07	.007	2.414	21.729

These discharges are converted to billions of cubic feet per month, shown in column three. After making allowance for evaporation losses from an assumed reservoir having 900 acres of water surface by use of columns four and five, the net monthly water supply in billions of cubic feet is shown in column six. The accumulated sums of the quantities in column six, shown in column seven, give the basis for the summation hydrograph which has the following characteristics :

1. The total run-off from the beginning of the record to any date is represented by the ordinate to the curve for that date.

2. The total run-off during any period of time is measured by the projection of the curve for that period on the run-off axis.

3. The rate of flow at any time is indicated by the slope of the curve at its intersection with the time ordinate if values of daily run-off are used in making the summation. To determine this rate, draw a line tangent to the curve at that point, extending it across the space for one

month. The difference in intercepts on the run-off axis at the beginning and end of month will give the total run-off in billions of cubic feet in a month if the rate were continued for that period. For convenience in determining rates of flow the diagram in the upper left hand corner of Plate VIII has been prepared with lines corresponding to various rates of flow. The slope of any part of the curve can be compared readily with this diagram and the rate of flow read directly or interpolated.

4. The average rate of flow for any period of time can be obtained by determining the slope of the line connecting the ends of the curve for that period as explained under 3.

5. The ordinate intercepted between the curve and a line connecting any two points on the summation hydrograph shows whether the total flow of the stream from the beginning of the period to the date indicated by that ordinate is greater or less than the total flow that would be produced during the same period by the rate of flow indicated by the slope of the straight line. If the ordinate is positive, *i. e.*, measured above the straight line, it shows the amount by which the total flow of the stream is greater than that produced by the flow of the draft line; if negative, it shows the amount by which it is less than the quantity produced by the flow corresponding to the slope of the draft line.

6. The amount of storage needed to equalize the flow for a given period of time can be determined from the summation hydrograph by connecting by a straight line the extremities of the hydrograph for the period and measuring on the scale of the ordinate axis the largest vertical distance between this line and the hydrograph.

Gerard H. Matthes[a] gives the following principal uses which he has made of the summation hydrograph:

1. Study of relations between storage and draft for power purposes on one and the same stream.

2. Same as before and in addition, relation with demands for irrigation interests below the power plant.

3. Regulations of a river for power purposes, with a reservoir not on the main stream but on a tributary.

4. Effect of regulating a river for the benefit of one power plant on the water supply of a similar power plant situated higher up the river but also below the reservoir.

5. The same problem as before, except that the reservoir is not on the river itself, but on a tributary.

6. Regulating a river to supply different rates of draft for different

[a] [illegible]

seasons of the year (it being assumed that in all the previous cases the draft was uniform).

7. For determining the economical size of a reservoir, be it for power purposes or for irrigation requirements, and consequent height of dam.

8. For studying the capacity of small terminal reservoir for peak load purposes.

The use of the summation hydrograph in determining the extent to which the flow of a stream may be regulated during any given period of time is illustrated in the following discussion of Plate VIII.

First consider the period January, 1910, to April, 1912. The average flow for this period is represented by the slope of the line AB which connects the ends of the curve for the period and corresponds to a uniform draft of 297 second-feet. The amount of storage necessary for equalizing the flow and thus maintaining a constant discharge of 297 second-feet during the period can be determined directly from the summation hydrograph by measuring on the scale of the ordinate axis the largest vertical distances s_1 and s_2 that the hydrograph is above and below the draft line AB. The sum of the ordinates s_1 and s_2 gives the minimum capacity of reservoir required for equalizing the flow. The ordinate s_1 represents a storage of two billion cubic feet and the ordinate s_2, five billion cubic feet. The required capacity of the reservoir is therefore seven billion cubic feet. It should be noted that the ordinate s_1, which is positive, *i. e.*, measured above the draft line AB, represents storage that can be supplied from the flow during the period under consideration, while the ordinate s_2, which is negative, *i. e.*, measured below the draft line AB, represents a storage requirement which can not be satisfied by flow from the beginning of period to the date indicated by the ordinate and for which water must be stored prior to that time if the assumed draft is to be maintained. In other words, from January, 1910, to September, 1911 (A to E), there was a total deficit of five billion cubic feet, and in order to make it possible to obtain a uniform flow of 297 second-feet during this period there should have been five billion cubic feet of water in the reservoir on January 1, 1910.

The distribution of the draft on this storage is shown by the storage curve, LMN, under the summation hydrograph. This curve is obtained by plotting the ordinates intercepted between the summation hydrograph and the draft line AB. Since it is necessary to have five billion cubic feet of water in the reservoir on January 1, 1910, the intercepted ordinates are plotted with reference to the storage line corresponding to five billion cubic feet. Positive ordinates are plotted above this line, and negative, below it. The storage curve shows:

1. That the five billion cubic feet of initial storage decreased to 4.6 billion cubic feet on February 28, 1910.

2. That the maximum storage in the reservoir occurred in March, 1910, and amounted to seven billion cubic feet.

3. That from April, 1910, to September, 1911, there was a draft on the storage until the reservoir was emptied in September, 1911.

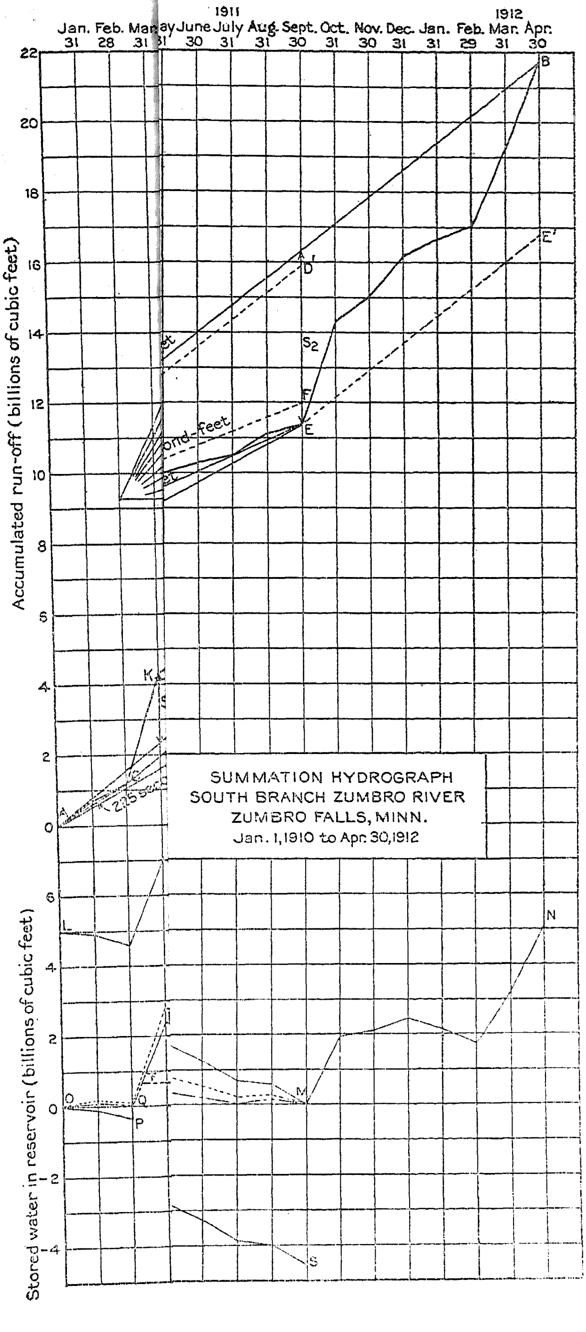
4. That after September, 1911, there was, in excess of the uniform flow of 297 second-feet, an accumulated storage amounting to five billion cubic feet in April, 1912, making the quantity of water in storage at the end of the period equal to that at the beginning.

The summation hydrograph shows that even though the total run-off from January 1, 1910, to April, 1912, was equivalent to an average flow of 297 second-feet, the distribution of the flow was such that even with an unlimited storage capacity the flow for the period could not have been regulated so as to yield 297 second-feet unless the reservoir contained five billion cubic feet of water on January 1, 1910, the beginning of the period under consideration.

If, with the reservoir empty on January 1, 1910, an attempt be made to obtain as nearly a uniform flow of 297 second-feet as is possible, the result would be as follows:

From January to February, 1910, the slope of the summation hydrograph is less than that of the draft line AB and, as the reservoir is assumed to be empty, it would be impossible to obtain a flow of 297 second-feet during that period. The most that could be obtained would be the actual flow of the stream which is equivalent to an average flow of 225 second-feet as shown by the slope of the line AC. At C the inclination of the summation hydrograph becomes greater than that of the draft line AB and the flow in excess of 297 second-feet will be stored in the reservoir. From C draw the draft line CD parallel to AB and extend it to D^1. During the period included between C and D, a uniform flow of 297 second-feet can be obtained. At D the reservoir will be empty and from this date until September, 1911, the flow is less than 297 second-feet. From October 1, 1911, to April, 1912, an average flow of 297 second-feet can be obtained and, in addition, there will be five billion cubic feet of water in the reservoir at the end of the period, as shown by the ordinate BE′ intercepted between B and the draft line EE′ which is drawn through E parallel to AB. The contents of the reservoir throughout the period is shown by that portion of the curve OPRSMN which lies above the zero line. The portions of the curve below the zero line indicate the periods and accumulated deficits when the flow was less than 297 second-feet. From Q to S this curve has been plotted

PLATE VIII.



SUMMATION HYDROGRAPH
SOUTH BRANCH ZUMBRO RIVER
ZUMBRO FALLS, MINN.
Jan. 1, 1910 to Apr. 30, 1912
Accumulated run-off (billions of cubic feet)
Stored water in reservoir (billions of cubic feet)
1911
1912
Jan. Feb. Mar. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr.
31 28 31 31 30 31 31 30 31 30 31 31 29 31 30

would occur is that included between the extremities of the draft line. In order not to require a storage greater than the capacity of the reservoir, the maximum ordinate intercepted between the draft line and the summation hydrograph must not exceed 0.664 billion cubic feet. This capacity is slightly exceeded in July and to a greater extent in January, 1911. It is therefore not possible to obtain a flow of 157 second-feet because of insufficient capacity of the reservoir. As a second trial lay off at G the ordinate GH equal to 0.664 billion cubic feet and from H draw a line tangent to the hydrograph at J. It is evident from inspection that the maximum ordinate intercepted between the hydrograph and HJ is HG, the capacity of the reservoir. Therefore from J to H it is possible to obtain a regulated flow represented by the slope of HJ equivalent to 147 second-feet. Through G draw a line parallel to HJ and extend it back to intersect the hydrograph at K. The flow during the period indicated by this line is sufficient to produce the regulated flow of 147 second-feet.

The regulated flow which may be obtained during the period G to E is represented by the slope of line GE which is equivalent to 166 second-feet. The storage required for this flow does not exceed the capacity of the reservoir, since the maximum ordinate intercepted between the draft line GE and the hydrograph does not equal the storage capacity of the reservoir.

The variation in the contents of the reservoir during the above regulation is shown by the storage curve which is shown by the broken line OQTM. After a slight storage in January, 1910, the reservoir became empty in February. During March the reservoir filled rapidly and remained full, water being wasted, until May. From this date the storage decreased until the reservoir became empty in January, 1911. Storage then increased to a maximum of about 0.39 billion cubic feet in March, 1911. The reservoir became empty in July and again in September, 1911.

ESTIMATING STREAM FLOW.

The engineer must often estimate the flow of streams of which few if any measurements of discharge have been made. The basis for such estimates of discharge and run-off may be (a) short time records in the basin; (b) records of precipitation and information in regard to other factors affecting stream flow; and (c) records from adjacent basins with which comparison may be made. At best such estimates are only roughly approximate and they should take account of all available information.

In planning a project depending for its success on surface waters, the collection of systematic records of flow should be begun as soon as possible, if such records are not already available. In general, the time required for bringing a project to the stage of financing and construction is sufficient to permit the collection of records adequate to serve as a basis for checking the estimates of water supply for the enterprise. Preliminary estimates will therefore be confirmed or refuted before any considerable investment is made.

The first attempt to present to engineers a rational basis for estimates of stream flow was made by F. H. Newell about 1890.[a] He prepared two curves showing a relation between rainfall and run-off, one for streams in mountainous regions, the other for streams draining basins characterized by broad valleys and gentle slopes. These curves show the relation indicated only in a general way and can not be safely relied on for estimating water supply. Many methods for estimating stream flow have since been outlined, the best being contained in a paper by Adolph F. Meyer [b]—"Computing run-off from rainfall and other physical data." In general, Mr. Meyer's method is designed for the extension of short time records of run-off by the use of longer records of climate. The extended estimates of run-off are obtained by subtracting from the recorded rainfall computed losses by evaporation from water, snow, ice, and land areas. A careful study of Mr. Meyer's paper is recommended to those who have occasion to estimate stream flow.

A comparison of short records of discharge in a basin, with long records of precipitation is of value in determining whether the available records of discharge represent conditions of high, low, or mean flow, even if no attempt is made to extend the actual record. The futility of estimating run-off by taking flat percentages of rainfall is illustrated by the tables on pages 160 to 163.

A simple method of estimating stream flow is to determine from records of other streams the probable discharge and run-off per square mile from the area under consideration. This, multiplied by the drainage area, gives the discharge. Such comparisons can be safely made, however, only when the streams used are situated in the same section of the country and are similar in size and character.

When few measurements are available, coefficients may be determined by means of which discharge may be estimated from the records for an adjacent drainage area. Plates IX and X show in a broad way the rainfall and run-off throughout the United States. As stated on page

[a] U. S. Geol. Survey, Fourteenth Ann. Report, 1892-3, pp. 149-153.

[b] Transactions American Society of Engineers, Paper No. 1348, Vol. LXXIX, page 1056 (1915).

156, from 19 to 28 inches of rainfall is required to satisfy evaporation and other losses. In areas in which rainfall is less than about 20 inches there is, as a rule, no run-off except during short periods of great precipitation, and for such areas only approximate estimates of run-off can be made. It should be noted that Plate X shows only general conditions and is not intended for use in estimating available water supply.

WHERE STREAM-FLOW DATA CAN BE FOUND.

As a result of studies by the Federal Government, by States, by special commissions, and by individuals, information in regard to stream flow and other water resources is now available for nearly all sections of the country This information is contained in publications that should form a part of every engineer's library and it should be freely used in order to avoid duplication of work.

The principal agencies that prepare and issue publications relating to water resources are—

1. United States Geological Survey.
2. United States Census.
3. United States Weather Bureau.
4. Corps of Engineers, United States Army.
5. State officials.
6. Special commissions.
7. City officials.

United States Geological Survey.—The United States Geological Survey has for many years carried on systematic measurements of flow of streams and publishes annually a report of the results of such measurements. In connection with this work it has made surveys of river profiles, studies of the quality of water, and investigations of related subjects, and from time to time has published special reports which either bring together all the data for particular drainage areas or discuss important hydrologic problems. Most of these reports are published in the series of water-supply Papers.

United States Census.—A report on the water powers of the important rivers of the United States was prepared and published in volumes 16 and 17 of the Tenth Census. During and since the Census of 1900 schedules have been prepared at each 5-year period showing the amount of water power utilized in the United States.

United States Weather Bureau.—Data in regard to precipitation,

evaporation, and other factors affecting the run-off of streams, are collected by the United States Weather Bureau, and published in the Annual Report of the Chief of the Weather Bureau, in the "Climate and Crop Reports," in "The Weather Review," and in special bulletins.

The Weather Bureau also maintains a "flood service," in connection with which records of daily fluctuations of river stage are collected at a large number of stations. These records have been printed under the title "Daily River Stages."

Corps of Engineers, United States Army.—The Army Engineers have investigated extensively the flow and slope of many of the larger rivers in the United States, including the Mississippi, Missouri, Niagara, and St. Lawrence. Data collected in these investigations are published in the annual reports of the Chief of Engineers and in reports of officers and special commissions working under the direction of the Chief of Engineers. The Army Engineers also have a large amount of manuscript data relative to the various streams.

State officials.—Much information has been collected and published by various States. In many States the State Engineer has charge of the collection and publication of the data; in others the work is carried on by the State Geologist or special commissions.

Special commissions.—Many problems relating to water resources have been investigated by commissions appointed by Federal, State, or city governments. Reports of such investigations are usually published and thus made available.

City officials.—Nearly all large cities have investigated and reported on local water supplies. These reports may usually be obtained by applying to the city engineer.

How to obtain Government publications.—Most Government publications may be obtained or consulted in the following ways:

(1) A small number of every report is delivered to the department under which the work was done. Copies of these reports may be obtained either free of charge or for a nominal sum by applying to the department publishing them.

(2) A certain number of each report issued is allotted to each member of Congress for personal distribution.

(3) Other copies are deposited with the Superintendent of Documents, Washington, D. C., from whom they may be purchased at cost of publication.

(4) Copies are furnished to the principal public libraries in the large cities throughout the United States, where they may be consulted.

REPORT WRITING.

The ability to write a clear, concise and comprehensive report contributes largely to the success of an engineer. Such ability, although relatively rare among engineers, can be acquired by giving proper attention to a study of (1) the purpose of a report, (2) the information to be included, (3) the method of presenting the information, and (4) the form of the report.

The duties of an engineer extend beyond his study of the physical features relating to an enterprise and include questions of administration, operation, economics, and finance, and even questions pertaining to the relation of the enterprise to the community. An engineering report, therefore, may and often must discuss all these related factors on which success may depend.

Broadly, the requirements of a successful reporting engineer are:

1. To see and evaluate possibilities.
2. To formulate features of design.
3. To estimate with reasonable accuracy the cost of construction.
4. To analyze and to appraise properly the market, industrial and social conditions.
5. To prepare a clear and concise statement covering the essential features of a project.
6. To draw sound and definite conclusions.

Stream-flow records and allied data form an important part of reports that discuss the use of water, and the engineer who collects or uses these data should therefore be able to present them clearly in a report containing complete information in regard to the project for which they have been compiled.

PURPOSE OF A REPORT.

Engineering reports may be divided into two classes—administrative and technical.

The object of an administrative report is to present information in regard to progress or status of investigations, development, or operation, in order that interested persons may be informed of its progress and that a permanent record may be made of the condition of the work at stated intervals of time.

A technical report may pertain to investigations of a project, to its development, to the operations of a going concern, or to a completed structure. Its object may be to present the important facts and conclusions pertaining to the physical or financial practicability of a project or to the economics of a going concern, for the consideration of persons

interested in the construction, operation, financing, or control of the enterprise; or the report may be made primarily to record permanently the information obtained.

INFORMATION TO BE INCLUDED IN A REPORT.

An administrative report should contain statements in regard to personnel, finances, progress of work, and like features, or to the factors and conditions affecting these features. If lengthy discussions of details are necessary, they should be presented in separate reports or appendixes.

Technical reports should contain statements of the technical and related features of the project or development and the conclusions derived from the statements. Every report should include—

1. An introduction stating the object of the report and giving a general description of the project or development and the sources of information.

2. A presentation, in the body of the report, of all important facts necessary to show the physical characteristics, feasibility, and estimated cost of the project, and its value when completed, as well as the elements of stability or of the risk involved, the nature of the presentation depending on the character of the enterprise. Complete statements relative to all factors affecting the project or development should be given, together with sufficient information to indicate the reliability of the data on which the conclusions rest.

3. The conclusions which should show concisely the results of the analysis of the data presented in the body of the report and the recommendations based on those conclusions. The report should be dated and signed on its final page, or a dated letter of transmittal, bearing the signature of the author, may be prepared.

Each report should include a title page, a table of contents, a list of illustrations, a list of tables, if necessary, and, if the report is long, an index. Long reports should be prefaced with an abstract of not more than two pages presenting the salient facts and conclusions. Related data or discussions not essential to a clear understanding of conditions but necessary as a basis for statements made in the report or for a detailed and critical analysis should be presented, if at all, in appendixes instead of in the body of the report.

As a basis for writing a report, an outline should be prepared and, to guard against omissions in estimates, a drawing of this or a similar enterprise showing every possible variation should be followed.

METHODS OF PRESENTING INFORMATION.

Information can be presented in three forms—text, tables, and illustrations.

All data to be used in the report should be carefully studied in order to determine which of these three forms affords the clearest and best method of presentation. Choice should be made primarily from considerations of conciseness and clearness, but the ability of the probable readers of the report to understand one or the other of these forms must also be considered.

Text.—The matter of the text should be presented in logical order and in simple and concise language. It should be divided into topics designated by center and if necessary by side headings under which the matter should be appropriately divided into paragraphs. References to information outside the report or to authorities cited should be made by footnotes. Citations of data within the report should be made by cross references, giving page numbers. Direct quotations should be exact as to wording, but errors in punctuation and other obvious printer's errors should be corrected. Proper credit for quotations, either direct or indirect, should be given either in the text or in foot-

ves.—Tables offer a convenient and effective method of presenting tical data and may also be used to present facts that are common to several units or groups, in order to disclose common or special characteristics or to make desirable comparisons. For example, the industrial or other features of the cities of a State may be presented more effectively by grouping them in tables under appropriate headings than by describing them in text. Tabular arrangement of information is illustrated in Plate VII.

All headings for tables should be clear and concise. There may be a choice not only as to the wording of headings of columns but as to their grouping as side heads or top heads. A proper choice of these headings may make it possible to combine two tables in one, or to present a table in more condensed and convenient form. A transposition of side and top heads may improve a table both in appearance and in clearness. Examples of the use of tables are shown in this book.

Each table should have an appropriate title and in some reports the numbering of tables may increase the ease and definiteness with which references may be made to them.

Illustrations.—Illustrations may be used to amplify the text or tables or as an independent means of presenting information. In general, they may be grouped in two classes—photographs and drawings. Pho-

tographs may show either general features or details of specific features. Drawings may be used to present data graphically or plans of features of the work, or, as maps, to show the locality and the positions of important features. A number and appropriate title should appear immediately below each illustration. The title of a photograph should always include the date on which it was taken.

FORM OF A REPORT.

All material in the report should be bound in regular book form. The first impression made by a report—a result of its general appearance—may determine its effect on the reader, and the ease with which it can be handled, read, and studied—a result of its general arrangement and make-up—may determine to a large extent its value and usefulness.

The manuscript should be typewritten on letter paper, with liberal margins and preferably with no visible corrections either by typewriter or pen. Except for quotations and tables, which may be single spaced, the lines should be double spaced. Pages should be numbered in the upper right hand corner. Tables and illustrations should be inserted in the text at or immediately after the place of first reference to them.

Not only the title page but the first page of text should bear the title of the report and the name of its author. A blank page should precede the title page and follow the last page of the report.

So far as possible, tables and illustrations should be reduced to the size of a page. If this is impossible, the sheet should not exceed twice the height of the page, as only one horizontal fold can be conveniently handled in a bound report. The length, however, is not thus limited as the bellows system of folding permits the ready use of several vertical folds.

So far as possible all drawings should be bound in the report but large sheets that must be folded horizontally more than once may be more conveniently used if placed in a pocket portfolio accompanying the report. A careful study of scales and a proper arrangement of matter may enable the writer to present information on sheets that may be bound in the report. Under no consideration should rolls of drawings accompany a report, as they are inconvenient both for handling and filing.

MEASUREMENT OF DRAINAGE AREAS FROM MAPS.

In many hydrologic studies it is necessary for the engineer to measure the areas of drainage basins. Tables accompany most planimeters, giving either (1) the proper settings so that the planimeter will give the area directly for maps of various scales, or, (2) the settings so that the readings will give square inches and coefficients to be used in con-

nection with maps of various scales for reducing these readings to square miles. In measuring drainage areas, however, it is more satisfactory to calibrate the planimeter, with the arm at any setting, directly from the map on which the area is to be measured rather than to use those tables. This method is applicable to maps constructed either on the Mercator or on the polyconic projection. By it considerable time is saved in making the measurement and greater accuracy is obtainable, as errors due to shrinkage or stretch of paper and those due to the planimeter itself, are eliminated.

The calibration is readily made by determining the number of revolutions of the planimeter wheel for a quadrangle of equal extent in latitude and longitude for which the area is given in standard tables similar to those shown on pages 143 and 144. The area at the given latitude corresponding to a revolution of the planimeter wheel for the map used, may then be determined by dividing the area of the measured quadrangle by the number of revolutions of the planimeter wheel, thus calibrating the instrument for that latitude and map.

In the calibration a quadrangle should be chosen, the middle parallel of which passes approximately through the center of gravity of the area. This is necessary in order to equalize the variation in area due to differences in latitude. In case the area extends over several degrees of latitude, it may be necessary to divide it into two or more parts and calibrate the planimeter for each part.

In determining an area it is necessary to measure only the portions which do not occupy full quadrangles as the areas for full quadrangles can be taken directly from the tables.

In using the planimeter, start at any observed wheel reading, without attempting to set the arm at zero. Move the pointer around the area in a clockwise direction and observe the final wheel reading. Change the position of the planimeter wheel on the paper, observe the initial reading and move the pointer around the area in a counter-clockwise direction and observe the final wheel reading. The differences between the initial and final readings in the two runs respectively should be very small and their mean will be the mean reading for the area. The double tracing of the area in this manner gives a check on the reading and when applied as explained removes the error due to lag of the instrument.

In some cases it is convenient to calibrate the planimeter, using the area of a State or county instead of the area of a quadrilateral. Areas of quadrilaterals of various sizes may be found in "Geographic Tables and Formulas," United States Geological Survey, from which the tables

Areas of quadrilaterals of the earth's surface of 30′ extent in latitude and longitude.

Middle latitude of quadrilateral.	Area in square miles.	Middle latitude of quadrilateral.	Area in square miles.	Middle latitude of quadrilateral.	Area in square miles.
° ′		° ′		° ′	
0 00	1,188.10	11 00	1,166.84	22 00	1,103.68
0 15	1,188.08	11 15	1,165.86	22 15	1,101.77
0 30	1,188.05	11 30	1,164.86	22 30	1,099.84
0 45	1,188.00	11 45	1,163.85	22 45	1,097.88
1 00	1,187.92	12 00	1,162.81	23 00	1,095.91
1 15	1,187.82	12 15	1,161.75	23 15	1,093.92
1 30	1,187.70	12 30	1,160.67	23 30	1,091.90
1 45	1,187.56	12 45	1,159.56	23 45	1,089.87
2 00	1,187.39	13 00	1,158.44	24 00	1,087.81
2 15	1,187.20	13 15	1,157.29	24 15	1,085.74
2 30	1,186.99	13 30	1,156.12	24 30	1,083.64
2 45	1,186.76	13 45	1,154.93	24 45	1,081.52
3 00	1,186.51	14 00	1,153.72	25 00	1,079.39
3 15	1,186.24	14 15	1,152.48	25 15	1,077.23
3 30	1,185.95	14 30	1,151.23	25 30	1,075.05
3 45	1,185.62	14 45	1,149.95	25 45	1,072.85
4 00	1,185.28	15 00	1,148.65	26 00	1,070.64
4 15	1,184.92	15 15	1,147.33	26 15	1,068.40
4 30	1,184.53	15 30	1,145.99	26 30	1,066.14
4 45	1,184.13	15 45	1,144.63	26 45	1,063.86
5 00	1,183.70	16 00	1,143.25	27 00	1,061.56
5 15	1,183.24	16 15	1,141.84	27 15	1,059.24
5 30	1,182.77	16 30	1,140.41	27 30	1,056.90
5 45	1,182.28	16 45	1,138.96	27 45	1,054.54
6 00	1,181.76	17 00	1,137.50	28 00	1,052.16
6 15	1,181.22	17 15	1,136.00	28 15	1,049.76
6 30	1,180.66	17 30	1,134.49	28 30	1,047.34
6 45	1,180.08	17 45	1,132.96	28 45	1,044.90
7 00	1,179.48	18 00	1,131.41	29 00	1,042.44
7 15	1,178.85	18 15	1,129.83	29 15	1,039.97
7 30	1,178.20	18 30	1,128.24	29 30	1,037.47
7 45	1,177.53	18 45	1,126.62	29 45	1,034.95
8 00	1,176.84	19 00	1,124.98	30 00	1,032.41
8 15	1,176.13	19 15	1,123.32	30 15	1,029.85
8 30	1,175.39	19 30	1,121.64	30 30	1,027.27
8 45	1,174.63	19 45	1,119.93	30 45	1,024.68
9 00	1,173.86	20 00	1,118.21	31 00	1,022.06
9 15	1,173.06	20 15	1,116.47	31 15	1,019.43
9 30	1,172.23	20 30	1,114.71	31 30	1,016.77
9 45	1,171.39	20 45	1,112.92	31 45	1,014.10
10 00	1,170.52	21 00	1,111.11	32 00	1,011.40
10 15	1,169.63	21 15	1,109.28	32 15	1,008.69
10 30	1,168.73	21 30	1,107.44	32 30	1,005.96
10 45	1,167.80	21 45	1,105.57	32 45	1,003.20

Areas of quadrilaterals of the earth's surface of 30′ extent in latitude and longitude (continued).

Middle latitude of quadrilateral.	Area in square miles.	Middle latitude of quadrilateral.	Area in square miles.	Middle latitude of quadrilateral.	Area in square miles.
° ′		° ′		° ′	
33 00	1,000.43	44 00	860.25	55 00	687.70
33 15	997.64	44 15	856.67	55 15	683.44
33 30	994.83	44 30	853.07	55 30	679.17
33 45	992.00	44 45	849.46	55 45	674.89
34 00	989.16	45 00	845.82	56 00	670.60
34 15	986.29	45 15	842.18	56 15	666.29
34 30	983.41	45 30	838.51	56 30	661.97
34 45	980.50	45 45	834.83	56 45	657.64
35 00	977.58	46 00	831.13	57 00	653.29
35 15	974.64	46 15	827.42	57 15	648.93
35 30	971.68	46 30	823.68	57 30	644.55
35 45	968.70	46 45	819.94	57 45	640.17
36 00	965.70	47 00	816.18	58 00	635.77
36 15	962.68	47 15	812.40	58 15	631.36
36 30	959.65	47 30	808.60	58 30	626.93
36 45	956.60	47 45	804.79	58 45	622.49
37 00	953.52	48 00	800.97	59 00	618.05
37 15	950.43	48 15	797.13	59 15	613.59
37 30	947.32	48 30	793.27	59 30	609.11
37 45	944.21	48 45	789.39	59 45	604.62
38 00	941.05	49 00	785.50	60 00	600.13
38 15	937.88	49 15	781.60	60 15	595.62
38 30	934.71	49 30	777.68	60 30	591.09
38 45	931.51	49 45	773.74	60 45	586.56
39 00	928.29	50 00	769.79	61 00	582.01
39 15	925.06	50 15	765.83	61 15	577.45
39 30	921.80	50 30	761.85	61 30	572.88
39 45	918.53	50 45	757.85	61 45	568.30
40 00	915.25	51 00	753.84	62 00	563.71
40 15	911.94	51 15	749.82	62 15	559.11
40 30	908.61	51 30	745.78	62 30	554.49
40 45	905.27	51 45	741.72	62 45	549.86
41 00	901.91	52 00	737.65	63 00	545.23
41 15	898.54	52 15	733.57	63 15	540.58
41 30	895.14	52 30	729.47	63 30	535.92
41 45	891.73	52 45	725.36	63 45	531.25
42 00	888.30	53 00	721.23	64 00	526.57
42 15	884.85	53 15	717.08	64 15	521.88
42 30	881.39	53 30	712.93	64 30	517.17
42 45	877.91	53 45	708.76	64 45	512.46
43 00	874.41	54 00	704.57	65 00	507.74
43 15	870.90	54 15	700.38	65 15	503.01
43 30	867.37	54 30	696.16	65 30	498.26
43 45	863.82	54 45	691.94	65 45	493.51

on pages 143 and 144 have been obtained. Standard areas of various States are given in Bulletin 302, United States Geological Survey, and areas of counties can be obtained from Rand and McNally Atlas maps.

Unfortunately, in measuring drainage areas in many sections of the country, maps of sufficient detail are not available for determining accurately the boundaries of the areas. In general, maps from various sources may be rated as to reliability in the following order:

(1) Results of special detail surveys.
(2) Topographic sheets, United States Geological Survey.
(3) United States Land Office maps.
(4) United States post route maps.
(5) Rand and McNally Atlas maps.
(6) Miscellaneous State and county maps.

LOGARITHMIC PLOTTING.[a]

In ordinary plotting, the co-ordinates or distances from the axes represent values of the variables. In logarithmic plotting, the co-ordinates represent values of the logarithms of the variables.

Thus figure 31 is the result of plotting directly the following simultan-

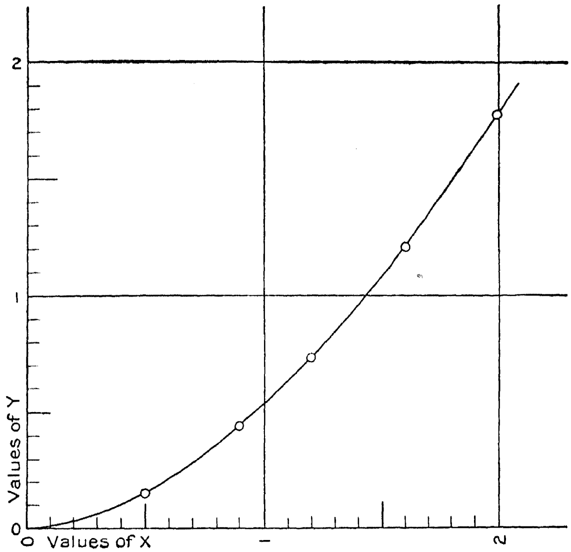


FIG. 31.

[a] "Hydraulic Laboratory Manual," by Professors Ernest W. Schoder and Kenneth B. Turner, Cornell University.

eous values of X and Y, the plotted points having been joined by a smooth curve:

X	Y
.50	.162
.90	.45
1.20	.74
1.60	1.22
2.00	1.80

Let us now tabulate the logarithms of the above values.

Log X	*Log Y*
9.6990(—10)	9.2095(—10)
9.9542(—10)	9.6532(—10)
.0792	9.8692(—10)
.2041	.0864
.3010	.2553

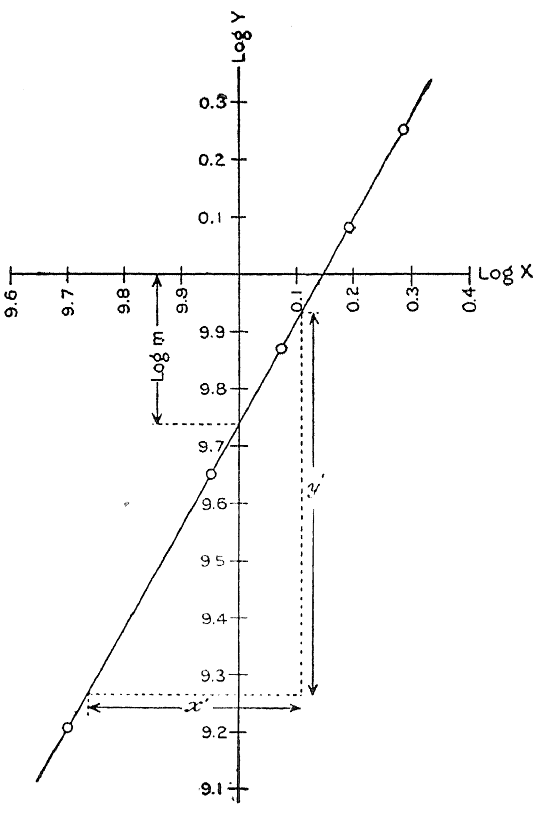


FIG. 32.

It should be noted that the logarithms of numbers less than 1 are negative. Instead, however, of writing the logarithm of 0.50 as —0.3010 (*i. e.* $log\ \frac{1}{2}=0-0.3010$), we change it to a whole negative number plus a positive decimal, *i. e.* either to $-1+0.6990$ (because $log\ \frac{5}{10}=0.6990-1$), written $\bar{1}.6990$, or to $9.6990-10$ (the -10 being usually omitted in writing, but always understood).

In figure 32 these simultaneous values of the logarithms are plotted. The numbers marked along the axes to the left of and below the origin are in accordance with the usual scheme of writing the negative logarithms.

In figure 32 the plotted points give a straight line, while in figure 31 with the direct plotting of the simultaneous values there is obtained a curve resembling a parabola. Herein appears one advantage of logarithmic plotting. In figure 31 we have no ready means of determining the equation of the curve, but in figure 32, since we have a straight line, the equation can be found readily as follows:

The equation of a straight line is of the form

$$y=ax+b \qquad (1)$$

where a is the slope of the line and b is the intercept on the Y axis, *i. e.* when $x=0$, $y=b$. So, by measuring the slope (the tangent of the angle made with the X axis) and the intercept, we can write out the correct equation of any straight line.

It is to be noted that the slope of a line may be negative as well as positive. If the line is in the second and fourth quadrants the slope is negative. Or, from another standpoint, when y increases with an increase in x the slope is positive and when y decreases with an increase in x the slope is negative.

Now, if we know, or assume, the equation

$$Y=mX_n \qquad (2)$$

we may write also

$$(log\ Y)=n(log\ X)+(log\ m) \qquad (3)$$

because if quantities are equal, their logarithms are equal.

Equation (3) is of the same form as (1), *i. e.* a straight line equation. In (3) the slope of the straight line is n and the intercept on the $(log\ Y)$ axis is $(log\ m)$ *i. e.* when $(log\ X)=0$, $(log\ Y)=(log\ m)$. Hence an equation like (2), which gives a parabola-like curve when corresponding values of X and Y are plotted, gives a straight line when the logarithms of X and Y are plotted. Conversely, when the logarithms of X and Y have been plotted, and the points found to lie on a straight line, we know that the equation is of the form

$$Y=mX^n$$

the slope of the line being equal to the exponent n and the intercept on the $(log\ Y)$ axis being equal to $(log\ m)$. To find m when $(log\ m)$ is known a table of logarithms is used.

The above reasoning holds good for all real values of the exponent n, whether positive or negative, whole number or fraction. m is assumed to be positive, as it usually is in equations that occur in engineering. We deal with only positive values of the original variables, X and Y, since the logarithm of a negative number is an imaginary quantity.

With the above facts demonstrated we can proceed to write the equation of the line in figure 32.

The slope is $\frac{y'}{x'}=1.75$. (See Fig. 32.) The intercept on the $(log\ Y)$ axis is negative and by the chosen scale the distance below the origin equals —0.277, or, by the system of representing negative logarithms, it equals 9.733(—10) as may be read directly on figure 32. Therefore the equation of the straight line is

$$(log\ Y)=1.75(log\ X)+(9.733-10).$$

Taking the anti-logarithms of both sides, we have

$$Y=0.54X^{1.75},$$

which is the desired equation in terms of X and Y, the original variables.

Let us now make use of *logarithmic scales* along the axes of figure 32, (See Fig. 33), and note the results.

On a logarithmic scale the divisions and marking are such that a division with some particular number represents (by its distance from the starting point) the logarithm of that number, just as on the common slide rule. Figure 34 shows an equal division scale and a logarithmic scale side by side. A careful study of these scales in their relation to each other will fix in mind the principle involved.

So by using the logarithmic scales it is not necessary, for instance, to scale off the intercept, as was done in figure 32, and then to look up the corresponding number in a table of logarithms. In figure 33 on the logarithmic scale at the left, horizontally opposite the intersection of the sloping line with the $(log\ Y)$ axis, we see the division representing 0.54. This is the same value of m previously obtained by the longer roundabout method.

So also in figure 33, opposite each plotted point, we see, on the left and bottom logarithmic scales, the divisions representing the values of X and Y given at the beginning of this discussion. It thus appears that the logarithmic scales enable us both to plot in proper position the logarithms of given numbers without using a table and also to read off

directly the number whose logarithm is represented by a given distance, *e. g.* an intercept.

Hence for purposes of logarithmic plotting we do not need the equal divisions along the axes, as in figures 32 and 33, but can advantageously

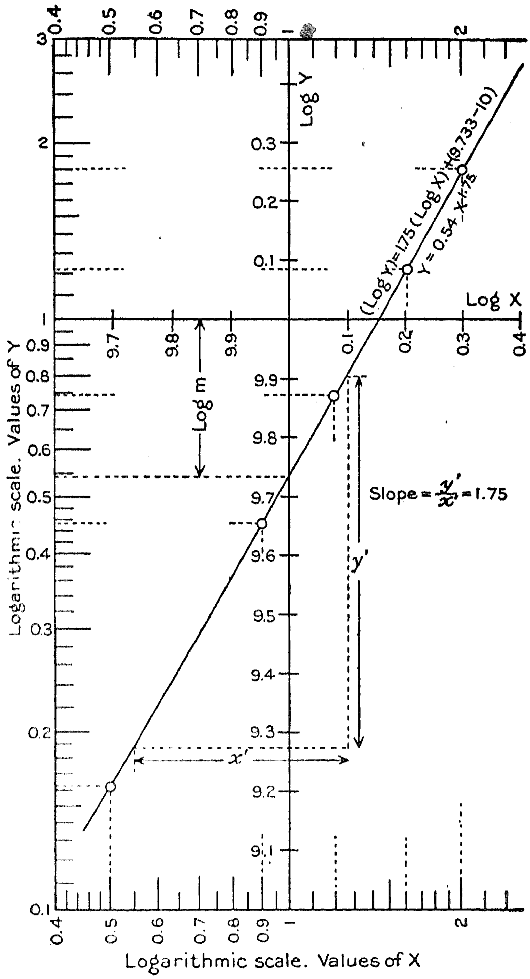


FIG. 33.

substitute the logarithmic scales. This brings us to the method of ruling *Logarithmic Cross-Section Paper* or a *Logarithmic Diagram.* As ordinary cross-section paper is made by drawing two sets of lines, equally spaced, perpendicular to each other, so logarithmic paper is made by

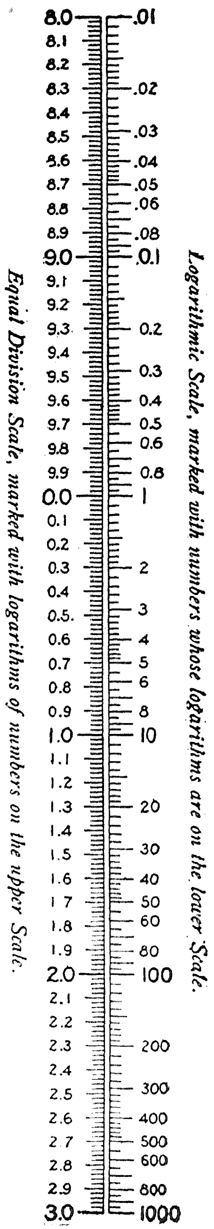


FIG. 34.

drawing two sets of lines spaced according to a logarithmic scale.

The "base" of such paper, and of a logarithmic scale in general, is the distance representing 1.0 in logarithms. Thus the "base" of the lower scales on the common 10-inch slide rule is 10 inches (sometimes 25 centimeters). The "base" of the upper scales of the slide rule is 5 inches.

A logarithmic scale has the same salient features as a common logarithmic table. Thus a table of logarithms contains the logarithms of all numbers between 1 and 10, advancing by intervals of, *e. g.*, .01 or .001 or .0001, etc. We consider such a table complete, but really it is not, because we modify the tabular logarithms by adding or subtracting one or more whole units (*the characteristic*) whenever the number is more than 10 or less than one. Under the same conditions we may consider a logarithmic scale to be complete when its divisions extend from 1 to 10. We can provide for the position of the decimal point by shifting one "base" length for each place that the decimal point is moved, because changing the decimal point one place on a number changes its logarithm by 1.0. Then on a logarithmic scale the position of the division representing the number would be moved one "base."

But in plotting we do not wish to bother with a scale that must be shifted about on the paper. The paper should be ruled so it will furnish its own scale at all points. Evidently, then, logarithmic scale cross-sections consist of a succession of panels one base square. All panels are ruled alike, just as on the upper scales of a slide rule the right half is a repetition of the left half. The logarithmic scale in figure 34 illustrates a succession of five base distances each divided alike, and giving a range of values from .01 to 1000.

It appears from figure 33 that the axes are situated where the logarith-

mic scales are marked 1. This is so because $(log\ 1)=0$. So on logarithmic paper the line for $(log\ X)=0$ is marked with the value of X, viz., 1. Therefore on logarithmic paper, in plotting lines from equations or finding equations from plotted lines, the origin always is at the intersection of the lines marked 1, and the intercept is to be taken on the Y axis, which is the line marked $X=1$.

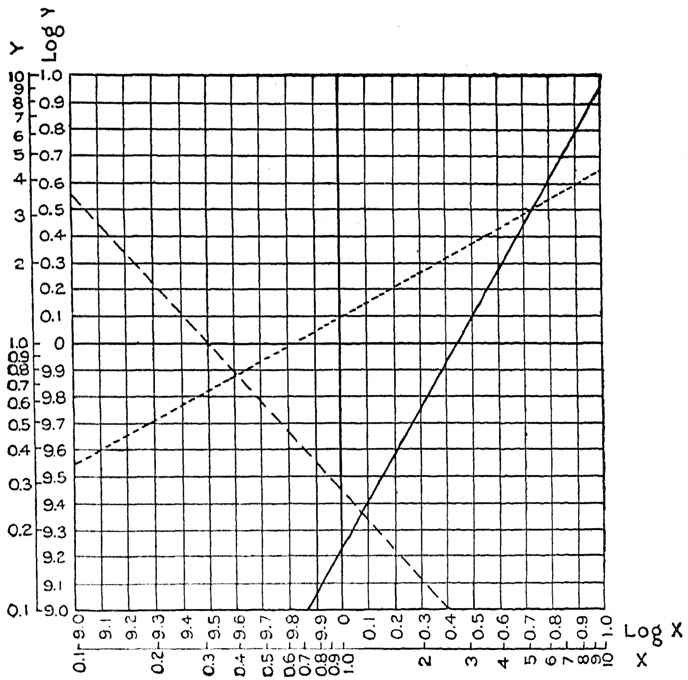


FIG. 35.

Of course on the unused sheet of logarithmic cross-section paper there is a division marked 1 every base distance along all four edges. We can choose any one of these for the unity value depending on convenience and the range of values it is desired to represent. After this we must properly place the decimal points on the printed numbers of the unused sheet. The lines one or more "bases" to the left or right of the Y axis represent $(log\ X)=-1, -2$, etc., or $+1, +2$, etc., and the markings are to be changed respectively, to 0.1, .01, etc., 10, 100, etc.

In short when we have selected an origin we must use it and no other during the whole calculations. Decimal points are not to be disregarded any more than on the slide rule.

In figures 35 and 36 are shown logarithmic plottings of three equations. The only difference between the figures is that the background in figure 35 is composed of equal division cross-section lines as on ordinary cross-section paper, while the background in figure 36 is composed of lines ruled according to a logarithmic scale as on logarithmic paper.

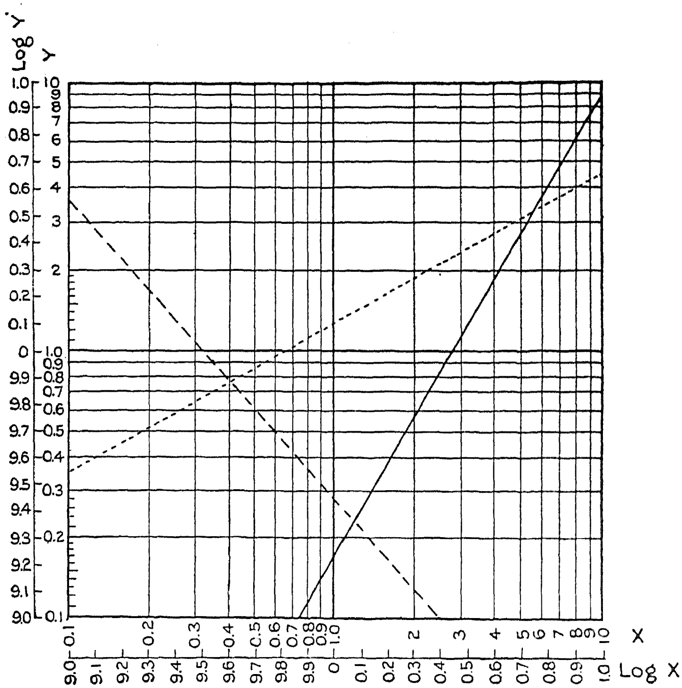


FIG. 36.

The equations in terms of X and Y, are respectively:

(Full line), $Y=0.166X^{1.72}$.

(Dotted line), $Y=1.26X^{0.55}$.

(Dashed line), $Y=0.282X^{-1.11}=\dfrac{0.282}{X^{1.11}}$.

These equations are read directly from figure 36, or indirectly from

figure 35 by first writing out the straight line equations of the logarithms, namely:

$$(log\ Y)=1.72\ (log\ X)+(9.22-10).$$
$$(log\ Y)=0.55\ (log\ X)+0.1.$$
$$(log\ Y)=-1.11\ (log\ X)+(9.45-10).$$

By taking the anti-logarithms of each side of these equations we obtain those given above. The advantage of using the logarithmic paper is obvious.

It is to be noted that the slope of a line on logarithmic paper is a ratio of two *distances*, and that these distances must be measured with an ordinary scale, *not with the logarithmic scale of the paper.*

Following are a few relations involving powers and roots, and in the representation of which logarithmic plotting is useful:

Flow in pipe:—

$$H=4f\,\frac{l}{d}\,\frac{v^2}{2g}, \qquad H=\frac{K}{d^{1.25}}\,V^{1.86},$$

$$V=C\,s^{.54}\,d^{.67}.$$

Flow in open channel:—

$$V=A\sqrt{Rs}.$$

Velocity of jet:—

$$V=C\sqrt{2gh}.$$

Head corresponding to velocity:—

$$h=\frac{V^2}{2g}.$$

Power in a nozzle stream:—

$$L=\frac{F\gamma V^3}{2g}$$

CHAPTER VI.

HYDROLOGY AS RELATED TO STREAM FLOW.

Water in its ceaseless round from atmosphere to earth and return, in its courses over or through the land or through animal or vegetable life, affords an endless number of interesting problems for study. It occurs in three forms—vapor, liquid and solid. It is distributed in the air as vapor or as clouds; on the surface of the earth, in running streams, ponds, lakes, the ocean, or frozen as ice or snow, and in plant and animal life; and in the ground as permanent or temporary ground water.

The science of hydrology relates to the occurrence of water in Nature. It includes a knowledge of the phenomena which pertain to and affect its appearance in the air, on the surface of the earth, and in the ground, together with its chemical and physical properties. The interrelation of the various phases of hydrology makes its study complex; consideration of any one phase necessarily involves practically the whole subject. In this discussion, which pertains principally to stream flow, there will be considered the conditions affecting the quantity and distribution of water from the time it reaches the earth in some form of precipitation until it flows into the ocean or is returned to the atmosphere. Knowledge of these conditions is necessary for the proper consideration of problems involving the economic utilization of water.

The water in surface streams is derived primarily from precipitation and represents that part of the precipitation that is left after evaporation, vegetation, seepage, and other losses have been satisfied. It reaches the streams either directly, by flow over the surface of the ground, or indirectly, by passage through the ground. The division of precipitation between surface and ground flow will depend largely on the intensity of precipitation. In general, the water of floods is derived from surface flow and that of medium and low stream stages from ground flow.

The factors that determine the quantity and distribution of water in streams include climate, vegetation, topography, geology, geographic location, and the works of man. Of these factors topography, geology and geographic location are practically permanent; the other three vary from time to time. The effects of these factors can not as a rule be differentiated, the laws governing them have not been fully developed,

and in general their magnitudes are not demonstrable; tendencies only can be discussed. They affect stream flow, however, in one or both of two ways, first, in the total yield and, second, in the seasonal distribution of run-off or the regimen; any factor may affect the flow directly or it may operate indirectly by its influence on one of the other factors.

It is noteworthy that whereas the climatic factors—especially precipitation and evaporation enter largely into the distribution of run-off, they also practically determine its total amount. The other factors, except as they affect climate, exert their influence principally on the distribution of flow and only slightly on the total quantity of discharge.

CLIMATE.[a]

Both the total flow of a stream and its distribution depend largely on climatic conditions. Precipitation, evaporation, temperature, wind, and humidity are the principal climatic factors.

In general, climatic conditions are determined chiefly by latitude, the relative distribution of land and water, the elevation of the land surface above sea level, and the prevailing winds which closely follow changes in barometric pressure.

PRECIPITATION.[b]

All water that appears in streams has at some time been condensed and precipitated from the atmosphere. The quantity, intensity, and distribution of precipitation are therefore principal factors determining the quantity and distribution of run-off. The effects of precipitation on stream flow are shown directly in the flow itself, but they are modified more or less by all the other conditions that affect stream flow.

For areas of considerable magnitude having sufficient precipitation to satisfy natural losses, the portion of precipitation available for stream flow depends on the magnitude of the losses through evaporation, vegetation, and seepage—losses that for a given locality are fairly constant and have been found to aggregate normally in the United States between 19 and 28 inches annually, the quantity depending on the length of the growing season and not on the amount of precipitation. Any locality in which the rainfall is less than is required for normal losses is nonproductive in stream flow except when excessive precipitation is temporarily greater than the losses.

Plates[c] IX and X illustrating graphically the average precipitation

[a] "Climatology of the United States," by Alfred Judson Henry, Bulletin Q., U. S. Weather Bureau, is an exhaustive treatise on this subject.

[b] Measurement of Precipitation, U. S. Weather Bureau Circular E, No. 445, describes methods of collecting records of both rainfall and snow.

[c] See Water-Supply Paper No. 301, U. S. Geol. Survey.

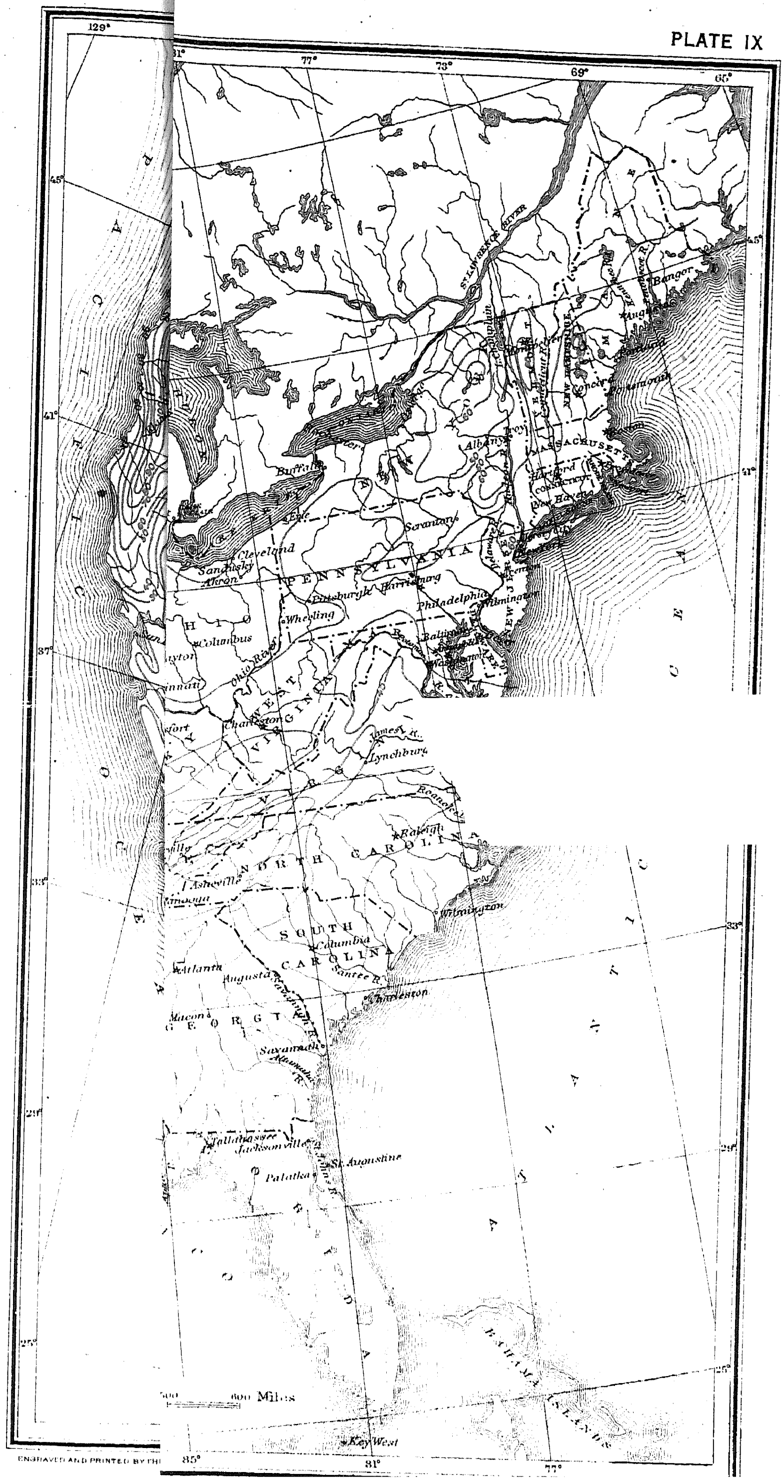
and the average run-off in the United States, show that the average run-off in most of the region east of the Appalachian range is between 20 and 30 inches. The rainfall in this area generally varies from 40 to 50 inches. West of the Appalachian Mountains, to the center of the Mississippi Valley, the run-off gradually decreases to about 10 inches and the rainfall to 30 inches. At the 99th parallel the run-off is about 3 inches and the rainfall less than 20 inches. From the 99th parallel to the Sierra and the Cascade ranges the rainfall, except on the high mountains, is less than 20 inches, and the annual run-off is less than 3 inches. In general, the streams in this region of low run-off gather their waters either from tributaries draining mountainous areas in which rainfall exceeds 20 inches or from areas of less rainfall during occasional periods of excessive precipitation. In the region between the Sierra and Cascade ranges and the Pacific Coast, except the floor of the Great Valley of California and the coastal area below San Francisco Bay, rainfall and run-off are higher than in any other part of the United States. In many places in this region both rainfall and run-off exceed 100 inches.

In preparing the map (Plate IX), showing lines of equal precipitation, rainfall data collected throughout the United States were supplemented and checked by stream-flow data for areas for which rainfall records were not available. When comparison of records showed a higher rate of stream flow than that indicated by the available rainfall data, the rainfall was estimated from the run-off data by adding 19 to 28 inches to the run-off.

In using rainfall records the conditions affecting their accuracy should be given careful consideration. The record obtained by a single rain gage shows only the measured precipitation on a few square inches of surface. This record, even if accurately made, may not be representative of a considerable area. In order to ascertain with reasonable certainty the average precipitation over a large area, many rain gages should be employed. Under ordinary conditions, however, the gages available in a given area are generally few and the extremes of precipitation, which always occur in comparatively small areas, may not be recorded. Rain gages are usually placed near habitations which in mountainous regions are generally at the lower elevations. Accurate records obtained by such gages may not therefore correctly represent the precipitation on the more elevated or less inhabited regions. It follows that the application of a few records to a large area may result in considerable error.

Satisfactory measurements of the snowfall of individual storms are

PLATE IX



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seldom obtained because of the difficulty in collecting in a receptacle a representative amount of snow falling in wind. In order to measure snowfall, therefore, it has been found most satisfactory to arrange a platform on which the snow is allowed to fall, and to collect and melt a vertical sample of known area, thus determining its water equivalent. The tube-and-scale method, devised by the United States Weather Bureau (Pl. XI, B), for determining the water equivalent of accumulated snow, represents the latest and best practice and will give satisfactory results when sufficient observations are made. The following table shows the record of accumulated snow and its water equivalent observed during an intensive study by this method on certain drainage areas in the White Mountains of New Hampshire. But few records of this character have been collected on account of the expense involved.

In considering rainfall data in connection with stream flow a record

Accumulated Snow and Water Equivalents on New Hampshire Drainage Basins.

Date.	Covert Brook.			Anderson Brook.			Shoal Pond Brook.			Burnt Brook.		
	Depth of snow.	Water equivalent.	Density.	Depth of snow.	Water equivalent.	Density.	Depth of snow.	Water equivalent.	Density.	Depth of snow.	Water equivalent.	Density.
1911	*Inches*	*Inches*		*Inches*	*Inches*		*Inches*	*Inches*		*Inches*	*Inches*	
Dec. 24 . .	3.3			6			8			6		
1912												
Jan. 1 . . .	4.7			6			9			10		
8 . . .	7.7	1.8	.23	10	2.7	.28	13	2.0	.15	13	2.1	.16
15 . .	15	2.5	.17	18	3.4	.19	22	3.4	.15	24	4.5	.19
22 . . .	19	4.1	.22	27	4.6	.17	29	5.1	.18	26	5.0	.19
29	19	4.5	.24	26	4.7	.18	28	5.3	.19	25	5.2	.21
Feb. 5 . .	22	5.1	.23	26	5.3	.20	31	5.9	.19	26	5.8	.22
12 . .	22	4.6	.21	26	5.2	.20	31	6.2	.20	26	5.4	.21
19 .	22	4.1	.19	25	4.7	.19	31	5.8	.19	28	5.9	.21
26 . . .	28	5.5	.20	31	6.3	.20	36	6.6	.18	37	5.5	.15
Mar. 4 . . .	35	5.8	.17	42	8.2	.20	46	8.5	.18	37	8.0	.22
11 .	34	5.5	.16	38	6.4	.17	43	7.1	.17	38	6.5	.17
18	30	5.4	.18	36	7.0	.20	43	6.7	.16	39	6.3	.16
25 . . .							45	9.8	.22	34	6.5	.19
Apr. 1 . . .	34	4.9	.14	37	7.9	.22	45	9.6	.21	37	7.9	.21
9 . . .	25	8.4	.34	30	8.8	.29	38	8.9	.23	30	7.0	.24
14 . .	22	8.0	.37	25	9.2	.37	34	10.6	.31	25	8.9	.36
18 . . .	15	5.0	.33	21	6.3	.31	27	8.0	.30	18	5.0	.28
22 .	12	5.1	.42	17	5.4	.32	21	6.2	.30	14	5.0	.36
29 . . .	7	.3	.43	14	4.5	.32	14	4.3	.26	6	2.1	.34

of the intensity of precipitation is as important as the total amount. Unfortunately comparatively few automatic rain gages have been operated, so that data in regard to intensity are lacking in most parts of the country. In using rainfall data it is necessary to assume that for any period of time the mean rainfall over the whole of an area is either the arithmetical or weighted mean of the rainfall during that period as

observed at the various stations in the area. As all rainfall records are liable to great errors, weighing is generally not warranted.

The records of precipitation show great variations from season to season and from place to place, with little if any ascertainable sequence or order. They show also great variations for different sections of the country and for different altitudes and exposures in the same sections. The mean yearly and seasonal rainfall for any locality is, however, fairly constant and has been determined for many observation stations from records extending over a series of years.

The average precipitation and the range of departure from the average has been determined with reasonable accuracy for many localities in the United States. Plate IX shows lines of equal rainfall drawn from the means of records collected in several years at many observation stations. The departures from the mean conditions can be determined for any place only by studying detailed records of precipitation.

In order to compare rainfall and run-off both records should be expressed in "depth in inches" over the drainage basins considered and should, of course, represent the same periods of time. Such data have usually been computed and recorded for calendar months. This period however, too short for purpose of comparison and may lead to apparently erroneous results, because heavy precipitation at the end of a month will not appear as run-off until the following month, as shown in the tables on pp. 160 to 163. A year is a better period but is not entirely satisfactory. The calendar year is undesirable as a comparative period, because the conditions of snow and ground storage are not the same at the end of every December. The year beginning with October or November, depending on the locality, is much better, as on the first of such period the conditions of storage are more nearly uniform from year to year, for at that time no snow is stored and the quantity of water held by the streams, lakes, and swamps and in the ground is usually at a minimum. The largest factor disturbing the relation between run-off and rainfall is storage in ground, surface, and snow, and as little information on this subject is available, it has been impossible to make proper allowance for its effect.

The tables[a] on pp. 160 to 163 show, for various drainage areas in northeastern United States, the monthly and yearly rainfall, run-off, and loss for each of the years for which run-off records are available. The records of precipitation were in some instances incomplete, and figures for several months in the period considered were missing. In such cases the mean of the records for the stations available was taken as the

[a] See Proceedings, American Society of Civil Engineers, Vol. XXXIII, May, 1907.

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MAP OF UNITED STATES, SHOWING MEAN ANNUAL RUN-OFF

Red lines and figures indicate average annual run-off in depth in inches

MAP OF UNITED STATES, SHOWING MEAN ANNUAL RUN-OFF

Red lines and figures indicate average annual run-off in depth in inches

mean for the month in question. Interpolation for supplying missing rainfall data adds nothing to the accuracy of the record and is probably never justifiable by theory or facts.

Rainfall records for the principal precipitation stations in the United States have been compiled and published by the U. S. Weather Bureau in 106 sections, under the title "Summary of Climatological Data for the United States."

EVAPORATION.

All precipitated water, in some part of its course over the earth's surface, is subject to evaporation which returns a portion of it to the atmosphere.

The effects of evaporation extend both to the total flow of streams and to the variations in flow during different seasons. The principal conditions on which the amount of evaporation depends are the temperature of the atmosphere and of the surface from which the evaporation takes place, the relative humidity of the air, and the wind movements. The relative importance of these effects has not been determined. The rate of evaporation from both land and water surfa
different localities and in the same locality in di
laws having general applicability have been discovered by which evaporation can be computed and, as with rainfall, information in regard to it has been obtained by direct observation.

But few direct measurements of evaporation from land surfaces have been made.[a] Indeed, it has generally been impossible to distinguish between the losses by vegetation and those by direct evaporation. In general the difference between rainfall and run-off from a given area gives the best available information as to the amount of such losses. As stated on page 155 these losses in the United States vary from 19 to 28 inches. The tables on pages 160 to 163 show the total losses for the northeastern portion of the United States.

Records of evaporation from water surfaces collected[b] at many points in the United States show an annual variation ranging from 20 to 40 inches in the humid eastern States and from 60 to 125 inches in the arid West. These records are of great value in studies of storage, as the total annual storage is diminished by the annual evaporation from the water surface.

The following table gives the monthly and annual evaporation at selected stations in the United States. The data in this table, with the

[a] See U. S. Geol. Survey Water Supply Paper No. 291.

[b] See reports of U. S. Weather Bureau for methods used and data collected.

Monthly and yearly maximum, minimum, and mean loss, for an

[Note:—M = Mean;

PRECIPITATION, IN INCHES.

Drainage.	October. M.	October. R.	November. M.	November. R.	December. M.	December. R.	January. M.	January. R.	February. M.	February. R.	March. M.	March. R.
Connecticut, above Orford; 3 300 sq. miles....		4.12		5.54		4.77		2.94		3.18		4.77
	2.99	2.12	2.17	1.05	2.80	1.36	2.28	1.99	1.60	0.71	3.44	2.05
Housatonic, above Gaylordsville; 1020 sq. miles		6.49		4.29		6.76		4.89		4.24		5.20
	3.98	2.74	2.39	0.89	4.38	2.72	3.05	1.65	2.56	0.76	4.10	3.04
Susquehanna, above Harrisburg; 24 080 sq. miles.		5.74		4.43		5.63		4.40		4.55		4.58
	3.02	0.95	2.63	0.92	2.97	1.04	2.73	1.77	2.67	0.98	3.35	1.21
Susquehanna, above Wilkes-Barre; 9 810 sq. miles..................		6.04		4.70		5.58		3.40		3.46		4.77
	3.38	1.69	2.41	1.13	3.30	2.24	2.57	1.69	2.80	1.17	3.60	3.17
Susquehanna, above Williamsport; 5 640 sq. miles		6.22		4.91		5.48		3.69		4.00		5.20
	2.90	0.89	2.74	0.54	3.15	1.25	2.63	1.51	2.72	1.05	4.09	3.42
Ohio, above Wheeling; 23 820 sq. miles.........		6.78		5.67		5.07		4.97		6.54		5.62
	2.66	0.51	3.09	0.65	3.13	1.84	3.24	1.72	3.19	1.19	3.39	1.40
Potomac, above Point of Rocks; 9 650 sq. miles..		6.41		4.10		5.71		3.78		5.88		4.44
	2.21	0.57	2.29	0.79	2.67	0.74	2.53	1.55	2.91	0.46	3.42	2.08
Shenandoah, above Millville; 3 000 sq. miles....		7.10		4.16		6.12		4.08		6.28		5.12
	2.47	0.48	2.08	0.81	2.61	0.23	2.58	1.41	3.12	0.33	3.52	2.08
James, above Cartersville; 6 230 sq. miles.........		8.35		3.57		7.19		4.43		5.08		6.38
	3.46	0.55	1.89	0.93	3.28	1.41	3.20	2.21	3.52	0.59	3.88	2.59
James, above Buchanan; 2 060 sq. miles..........		8.07		5.20		7.63		4.56		5.80		5.66
	2.52	0.46	2.42	0.71	3.00	0.30	2.79	1.77	3.66	0.63	3.88	2.39
James, above Glasgow; 830 sq. miles...........		8.70		5.27		7.60		4.49		5.48		6.36
	2.61	0.32	2.29	0.73	2.87	0.17	2.86	1.80	3.57	0.49	3.89	2.67
Appomattox, above Mattoax; 745 sq. miles......		7.60		3.99		8.08		4.82		5.22		5.96
	2.70	0.35	2.20	1.06	3.76	1.88	3.28	2.00	3.11	0.94	3.63	2.37
Roanoke, above Roanoke; 390 sq. miles............		6.32		4.60		7.45		4.08		7.10		6.49
	2.71	0.13	2.49	1.04	2.92	0.50	2.85	1.64	3.92	0.62	4.01	2.44
Roanoke, above Randolph; 3 080 sq. miles...		5.54		3.15		7.84		4.35		5.63		6.49
	2.61	0.65	2.22	1.22	3.95	1.78	3.10	2.19	3.56	0.88	3.57	2.29

RUN-OFF, IN INCHES.

Drainage.	October. M.	October. R.	November. M.	November. R.	December. M.	December. R.	January. M.	January. R.	February. M.	February. R.	March. M.	March. R.
Connecticut, above Orford....................		1.94		2.51		2.34		1.11		1.03		7.56
	1.24	0.45	1.23	0.50	1.28	0.47	0.76	0.27	0.54	0.26	3.91	1.60
Housatonic, above Gaylordsville		3.25		1.92		4.13		3.21		(3.68)		8.77
	1.89	(0.40)	1.39	0.96	2.68	(1.03)	2.32	0.98	1.61	0.49	5.88	(4.24)
Susquehanna, above Harrisburg		2.17		2.18		3.53		3.79		4.04		7.46
	0.90	0.16	1.08	0.28	1.75	0.49	1.94	0.67	1.98	0.53	4.48	2.46
Susquehanna, above Wilkes-Barre..........		3.22		1.47		4.91		3.45		3.92		7.84
	1.16	0.13	0.96	0.60	2.53	0.90	2.89	2.14	2.56	1.57	5.33	2.79
Susquehanna, above Williamsport..............		2.68		1.84		4.14		3.23		4.52		8.09
	0.85	0.15	1.14	0.29	1.75	0.33	1.96	1.01	1.96	0.56	5.59	2.82
		3.70		2.97		3.64		4.30		7.29		6.89
Ohio, above Wheeling....	0.72	0.12	1.21	0.24	1.99	0.53	2.78	1.28	3.12	0.78	4.07	1.37
Potomac, above Point of Rocks..................		1.63		0.99		3.06		2.49		4.60		6.50
	0.52	0.14	0.44	0.15	1.06	0.26	1.30	0.51	2.00	0.39	2.80	1.34
Shenandoah, above Millville....................		2.99		1.05		3.12		2.62		3.63		5.34
	0.82	0.20	0.48	0.20	1 05	0.29	1.21	0.47	1.53	0.46	2.16	0.86
James, above Cartersville....................		2.34		1.27		3.29		2.76		3.74		5.29
	0.89	0.21	0.75	0.26	1.51	0.46	1.80	0.68	2.11	0.64	3.16	1.52
		2.33		2.46		4.82		2.51		5.34		5.67
James, above Buchanan.	0.60	0.18	0.71	0.20	1.30	0.26	1.36	0.42	2.34	0.51	3.35	1.53
		2.48		1.67		4.50		2.79		3.99		4.44
James, above Glasgow...	0.67	0.22	0.63	0.24	1.21	0.31	1.41	0.63	2.34	0.43	2.82	1.28
Appomattox, above Mattoax....................		1.44		1.32		2.54		2.73		3.60		4.01
	0.63	0.27	0.64	0.33	1.46	0.58	1.75	0.61	2.29	0.45	2.26	0.78
		3.06		1.71		4.23		3.34		5.63		7.49
Roanoke, above Roanoke.	0.85	0.26	0.78	0.24	1.29	0.35	1.43	0.22	2.34	0.64	2.88	0.94
Roanoke, above Randolph..................		1.82		1.08		3.61		2.29		3.49		4.13
	1.03	0.30	0.81	0.32	1.71	0.71	1.63	0.78	1.98	1.02	2.33	1.04

rainfall, run-off in percentage of rainfall, and average year.

R = Range.]

April.		May.		June.		July.		August.		September.		Year.		No. of Years.
M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	Total.	R.	
2.77	3.54 1.37	2.99	4.78 0.27	3.78	5.27 2.10	4.34	5.06 3.76	3.88	4.59 3.22	3.73	5.75 1.08	36.76	41.80 33.48	5
3.71	6.12 2.25	2.97	6.24 1.12	5.46	10.42 1.86	5.00	7.25 3.72	5.56	7.28 3.45	4.70	6.42 1.94	47.86	51.49 39.77	5
2.76	4.46 1.27	3.96	7.70 1.27	3.98	6.44 2.77	4.11	7.24 2.42	4.16	6.48 1.92	3.04	5.61 1.41	39.38	45.17 31.62	14
2.70	4.67 1.50	2.73	5.39 1.11	4.46	6.38 2.94	5.05	7.86 4.03	4.49	6.51 2.73	2.90	4.32 1.40	39.85	44.13 31.77	6
2.89	4.69 1.33	3.20	5.41 1.74	4.11	6.03 2.94	4.62	7.58 2.77	4.14	6.62 2.26	2.83	4.70 1.05	40.02	44.11 33.04	10
3.28	6.50 1.57	4.04	7.48 2.18	4.32	5.80 2.50	4.55	9.08 2.54	3.74	6.88 1.30	3.07	6.48 1.56	41.71	55.56 33.47	21
2.61	6.05 1.34	3.77	6.47 1.97	4.15	6.57 1.81	4.15	6.63 2.28	3.50	7.00 1.69	2.65	6.09 1.32	36.86	44.81 29.37	10
2.55	6.24 1.16	3.85	5.82 2.28	4.90	7.63 2.09	4.14	6.21 2.17	3.56	7.73 1.41	2.95	7.22 1.01	38.33	48.08 30.47	10
3.07	6.92 1.72	3.75	6.70 1.78	5.13	7.73 3.55	4.06	7.47 2.35	4.50	10.22 1.54	3.24	4.11 1.98	42.98	54.83 30.58	7
2.66	6.52 1.57	4.20	6.31 1.25	4.78	7.57 3.34	4.42	8.48 2.27	3.67	8.71 1.61	3.17	6.20 1.06	41.17	53.31 30.45	10
2.70	7.08 1.19	4.04	6.18 1.33	4.78	8.71 2.72	4.09	6.22 2.18	3.82	7.47 1.46	3.24	6.70 0.73	40.76	51.48 32.48	10
3.09	5.99 1.08	3.96	7.26 1.72	3.99	5.04 3.20	4.13	7.05 1.94	6.24	13.06 2.70	2.89	4.20 2.29	42.98	52.98 30.80	5
2.80	6.50 1.67	4.18	7.46 0.93	4.77	8.14 1.90	4.91	11.64 3.08	3.80	10.72 0.98	3.32	5.16 1.22	42.68	58.30 35.19	9
3.35	6.04 1.48	4.07	6.33 1.92	4.53	5.98 2.83	4.92	5.09 2.68	5.15	11.21 2.40	2.77	3.29 1.86	43.80	53.95 34.00	5

April.		May.		June.		July.		August.		September.		Year.		No. of Years.
M.	R.	L.	R.	M.	R.	M.	R.	M.	R.	M.	R	Total.	R.	
4.70	7.10 3.64	3.10	4.80 1.16	1.69	3.20 1.02	1.09	1.51 0.49	1.09	1.53 0.69	1.03	1.82 0.37	21.66	27.04 16.01	5
4.68	6.43 3.82	2.40	4.50 1.11	2.24	4.28 0.92	1.47	2.49 0.56	1.41	2.19 0.87	1.51	2.25 1.08	29.43	36.94 23.76	5
3.48	4.83 2.34	2.07	4.54 0.61	1.25	3.03 0.50	0.83	3.25 0.34	0.77	1.60 0.24	0.61	1.42 0.17	21.09	28.03 16.34	14
3.17	4.46 2.19	1.13	2.52 0.40	1.07	1.79 0.10	1.01	3.41 0.28	0.66	1.53 0.12	0.72	1.44 0.15	23.19	27.18 15.15	6
3.50	5.45 2.24	1.68	3.15 0.60	1.20	2.41 0.51	1.23	4.11 0.36	0.87	1.41 0.27	0.52	1.24 0.18	22.26	27.60 16.57	10
3.20	6.84 1.80	1.94	5.10 0.51	1.30	3.37 0.31	1.06	3.49 0.28	0.76	1.88 0.16	0.53	2.50 0.15	22.68	34.20 16.29	21
1.98	4.60 0.76	1.34	3.22 (0.31)	0.99	2.18 0.37	0.76	1.52 0.29	0.69	2.66 0.23	0.34	0.88 0.16	14.22	21.46 8.16	10
1.77	4.79 0.72	1.39	3.86 0.53	1.16	3.07 0.52	0.83	1.71 0.34	0.81	3.15 0.38	0.43	0.93 0.23	13.64	19.78 7.86	10
2.18	4.45 1.01	1.63	3.45 0.94	1.50	3.05 0.68	0.99	1.45 0.38	1.02	3.08 0.30	0.67	1.25 0.29	18.21	24.73 10.69	7
2.08	4.98 0.88	1.77	3.55 0.58	1.17	3.15 0.49	0.98	3.02 0.24	0.81	2.71 0.22	0.50	0.99 0.21	16.91	26.30 11.45	10
1.79	4.20 0.80	1.51	2.83 0.58	1.15	3.00 0.30	0.99	2.82 0.24	0.84	2.49 0.25	0.63	2.14 0.17	15.99	21.33 12.15	10
2.13	3.67 0.85	1.44	3.19 0.92	0.90	1.51 0.48	0.73	1.31 0.39	1.42	4.01 0.53	0.83	1.18 0.37	16.48	25.15 10.92	5
1.89	4.90 0.58	1.79	4.36 0.76	1.14	2.54 0.54	1.16	3.54 0.39	1.33	5.73 0.26	0.80	1.53 0.22	17.69	29.66 8.88	9
1.88	3.49 0.81	1.65	3.16 1.10	1.37	1.73 1.05	1.45	2.43 0.79	1.80	4.94 0.82	1.02	1.45 0.65	18.66	25.16 10.99	5

Monthly and yearly maximum, minimum, and mean loss, for an

[Note:—M = Mean;

RUN-OFF IN PERCENTAGE OF RAINFALL.

Drainage.	October.		November.		December.		January.		February.		March.	
	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.
Connecticut, above Orford	41	92 17	57	132 33	46	108 17	33	55 13	34	86 23	114	163 45
Housatonic, above Gaylordsville	47	64 (13)	58	175 22	61	67 (87)	76	149 (41)	63	(87) (39)	143	203 87
Susquehanna, above Harrisburg	30	44 6	41	105 11	59	100 19	71	133 32	74	209 41	134	277 60
Susquehanna, above Wilkes-Barre	35	58 4	40	85 22	77	140 40	112	201 76	111	197 (47)	148	223 78
Susquehanna, above Williamsport	29	64 10	42	68 12	56	102 15	75	116 50	72	146 43	137	200 71
Ohio, above Wheeling	27	72 6	39	114 12	64	99 27	86	129 46	98	151 51	120	191 82
Potomac, above Point of Rocks	24	162 8	19	41 6	40	76 10	51	98 30	69	168 33	82	149 41
Shenandoah, above Millville	33	623 10	23	64 12	40	383 9	47	78 (27)	49	139 (26)	61	148 23
James, above Cartersville	25	131 13	36	69 23	46	70 21	56	74 31	60	108 34	81	128 44
James, above Buchanan	24	92 8	29	75 14	43	67 11	49	75 24	64	105 16	86	172 40
James, above Glasgow	26	381 8	28	64 11	42	653 12	49	78 34	66	168 51	70	127 37
Appomattox, above Mattoax	28	160 11	29	47 13	39	71 17	53	77 30	74	88 48	62	76 26
Roanoke, above Roanoke	31	208 9	31	77 15	39	92 7	50	90 13	60	152 15	72	115 27
Roanoke, above Randolph	40	180 25	36	89 17	43	53 24	53	71 36	56	116 (31)	65	103 41

LOSS, IN INCHES.

Drainage.	October.		November.		December.		January.		February.		March.	
	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.
Connecticut, above Orford	1.75	2.60 0.18	0.94	3.03 0.41	1.52	2.43 -0.06	1.52	1.97 0.91	1.06	2.15 0.10	-0.47	1.92 2.91
Housatonic, above Gaylordsville	2.09	3.24 0.98	1.00	3.33 -0.67	1.70	2.63 0.96	0.73	1.99 -1.05	0.95	2.36 (0.20)	-1.78	0.66 -4.46
Susquehanna, above Harrisburg	2.12	4.16 0.66	1.55	3.42 -0.11	1.22	2.76 0.00	0.79	1.84 -0.62	0.69	2.14 -1.21	-1.13	1.65 3.58
Susquehanna, above Wilkes-Barre	2.17	3.45 1.27	1.45	3.46 0.17	0.77	1.70 -0.93	-0.32	0.83 -1.72	-0.26	0.65 -1.93	-1.73	0.80 -4.83
Susquehanna, above Williamsport	2.05	4.71 0.46	1.60	3.58 0.21	1.40	2.82 -0.03	0.67	1.50 -0.39	0.76	1.92 -0.84	1.50	1.02 4.05
Ohio, above Wheeling	1.94	3.96 0.39	1.88	4.76 -0.37	1.14	2.25 0.03	0.46	1.91 -0.88	0.07	0.90 -1.28	-0.68	0.69 3.13
Potomac, above Point of Rocks	1.68	4.78 0.57	1.85	3.86 0.64	1.61	2.65 0.18	1.23	2.09 0.05	0.91	2.32 -0.75	0.62	2.60 2.15
Shenandoah, above Millville	1.65	4.91 -2.51	1.60	3.48 0.29	1.56	3.00 -0.65	1.37	2.11 0.55	1.59	(3.67) -0.13	1.36	(2.40) -1.73
James, above Cartersville	2.57	(6.01) -0.17	1.14	2.67 0.29	1.77	3.90 0.95	1.40	1.93 0.83	1.41	2.31 -0.05	0.72	1.96 0.92
James, above Buchanan	1.92	2.85 0.20	1.71	3.83 0.50	1.70	3.07 0.71	1.43	2.05 0.72	1.32	2.66 -0.03	0.53	2.62 2.27
James, above Glasgow	1.94	6.22 -0.90	1.66	3.60 0.42	1.67	3.10 -0.94	1.45	1.88 0.86	1.23	2.30 -1.34	1.07	2.55 -0.86
Appomattox, above Mattoax	2.07	6.16 -0.21	1.56	2.67 0.56	2.30	5.77 0.86	1.52	2.09 0.76	0.82	1.62 0.40	1.38	2.27 0.71
Roanoke, above Roanoke	1.86	3.96 -0.14	1.71	3.91 0.43	1.63	3.33 0.04	1.42	2.61 0.39	1.57	4.10 -0.32	1.13	2.53 1.00
Roanoke, above Randolph	1.58	3.72 -0.56	1.41	2.13 0.14	2.24	4.23 1.05	1.47	2.06 0.82	1.58	2.73 -0.14	1.24	2.36 -0.09

rainfall, run-off in percentage of rainfall, and average year.

R = Range.]

April.		May.		June.		July.		August.		September.		Year.		No. of Years.
M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	Mean.	R.	
170	266 120	104	430 65	45	68 20	25	38 13	28	30 16	28	34 22	59	65 46	5
125	187 105	81	118 72	41	113 20	29	61 11	25	40 18	32	80 17	62	73 58	5
124	324 25	52	72 21	31	86 10	20	47 9	18	47 6	20	73 5	55	68 44	14
117	200 96	41	50 26	24	58 10	20	43 7	15	24 4	25	74 11	58	65 47	6
121	276 85	52	69 34	29	66 12	27	54 12	21	82 7	18	56 7	56	68 42	10
98	156 58	48	82 18	30	67 10	23	53 9	20	56 5	17	39 6	54	68 44	21
76	104 35	36	57 (11)	24	48 11	18	34 9	20	38 10	13	28 4	39	61 22	10
69	118 31	36	65 14	24	41 13	20	43 11	23	41 10	15	41 5	36	58 21	10
71	101 48	48	66 32	29	54 18	24	37 12	23	30 11	21	44 8	42	50 33	7
76	128 40	42	79 19	25	52 14	22	38 8	22	39 5	16	80 8	41	58 28	10
66	128 37	37	69 18	24	52 9	24	33 10	22	33 6	19	32 6	39	53 33	10
69	113 42	36	54 24	23	41 13	18	32 9	23	31 14	29	49 14	38	48 30	5
66	89 25	43	125 75	24	57 14	24	48 12	35	58 8	24	37 7	41	56 25	9
56	87 31	41	66 23	30	38 21	29	46 16	35	44 24	37	44 25	43		

April.		May.		June.		July.		August.		September.		Year.		No. of Years.
M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	Total.	R.	
—1.94	—0.70 —3.62	—0.11	1.67 —1.39	2.09	4.23 1.05	3.25	3.81 2.41	2.79	3.62 2.00	2.70	3.74 0.71	15.10	18.81 12.84	5
—0.92	0.20 —2.20	0.57	1.74 —0.22	3.22	6.14 —0.25	3.53	5.06 1.56	4.15	5.71 2.08	3.19	5.19 0.39	18.43	22.56 13.39	5
—0.67	1.37 —2.81	1.89	3.16 0.58	2.73	5.57 0.40	3.28	4.41 1.94	3.39	5.25 1.22	2.43	4.81 0.53	18.29	21.04 13.54	14
—0.46	0.21 —1.50	1.60	2.87 0.71	3.39	5.30 1.32	4.04	4.58 3.47	3.83	5.10 2.53	2.18	3.76 0.43	16.66	18.61 14.52	6
—0.61	0.17 —2.54	1.52	2.26 0.91	2.91	5.05 1.25	3.39	4.58 2.41	3.27	5.18 0.96	2.31	4.39 0.87	17.76	20.89 15.70	10
0.07	2.00 —1.56	2.10	3.14 0.95	3.02	4.91 1.33	3.50	6.07 2.26	2.98	5.44 1.09	2.53	4.50 0.81	19.02	24.86 15.18	21
0.63	1.60 0.14	2.43	3.77 1.26	3.16	4.49 1.33	3.39	5.41 1.98	2.81	5.23 1.27	2.31	5.84 1.03	22.61	29.09 13.87	10
0.78	1.82 —0.30	2.46	3.97 1.46	3.71	5.14 1.49	3.31	5.11 1.73	2.75	4.58 1.03	2.52	6.83 0.76	21.69	33.05 14.58	10
0.89	2.47 0.02	2.12	3.59 0.60	3.63	5.37 2.57	3.07	4.98 1.91	3.48	7.14 1.24	2.57	3.54 1.33	21.77	30.79 18.90	7
0.64	1.54 —0.37	2.43	4.10 0.26	3.60	5.77 2.79	3.44	5.46 1.87	2.86	6.00 1.30	2.67	5.21 0.83	24.26	32.38 11.89	10
0.91	2.88 —0.45	2.53	3.47 0.41	3.68	6.87 1.92	3.10	4.18 1.91	2.97	4.98 1.15	2.61	4.56 0.50	24.77	30.46 16.29	10
0.95	2.51 0.41	2.52	4.07 0.79	3.09	4.04 2.19	3.40	6.42 1.51	4.82	9.05 2.17	2.07	3.46 1.22	26.50	36.10 19.88	5
0.91	1.70 0.20	2.89	4.91 —0.23	3.63	6.85 0.81	3.75	8.10 2.44	2.47	4.99 0.72	2.53	4.81 0.84	24.99	31.71 15.91	9
1.47	2.55 0.24	2.42	4.95 0.65	3.16	4.75 1.78	3.47	6.30 1.87	3.35	6.27 1.58	1.75	1.99 1.21	25.14	29.38 16.00	5

exception of those collected at Chestnut Hill Reservoir, Boston, Mass., and at Mount Hope Reservoir, Rochester, N. Y., were obtained from publications of the U. S. Weather Bureau, and are the results of a year's observations made during 1909–1910.

Monthly Evaporation at Different Points in the United States.

Month	California, Ohio, floating pan diameter—four feet		Birmingham, Ala., floating pan diameter—four feet		Chestnut Hill Reservoir, Boston Mass., floating pan		Mt. Hope Reservoir, Rochester, N. Y., floating pan		Dutch Flats Neb., ground pan diameter—four feet		Deer Flat, Idaho, ground pan diameter—three feet	
	Evaporation	Per cent of yearly	Evaporation	Per cent of yearly	Evaporation	Per cent of yearly	Evaporation	Per cent of yearly	Evaporation	Per cent of yearly	Evaporation	Per cent of yearly
January	*1.00	2.2	*1.50	2.9	0.96	2.4	0.52	1.5	*1.75	2.7	*1.50	1.9
February	*1.50	3.3	*1.50	2.9	1.05	2.7	.54	1.6	*1.75	2.7	*2.25	2.8
March	*2.50	5.4	*2.25	4.4	1.70	4.3	1.33	3.9	*3.00	4.6	*4.00	5.1
April	4.12	9.0	4.45	8.6	2.97	7.6	2.62	7.6	*4.50	6.9	*7.25	9.2
May	5.07	11.0	5.91	11.5	4.46	11.4	3.93	11.4	*6.25	9.4	10.68	13.5
June	6.21	13.5	7.28	14.2	5.54	14.2	4.94	14.3	8.05	12.2	11.05	14.0
July	7.20	15.6	7.36	14.4	5.98	15.2	5.47	15.8	10.95	16.7	11.15	14.1
August	7.26	15.8	7.34	14.3	5.50	14.0	5.30	15.4	9.39	14.3	11.77	14.9
September	5.63	12.2	6.00	11.7	4.12	10.4	4.15	12.0	7.44	11.3	*9.75	12.3
October	*3.00	6.5	*4.00	7.8	3.16	8.1	3.16	9.1	5.59	8.5	5.40	6.9
November	*1.50	3.3	*2.25	4.4	2.25	5.7	1.45	4.2	*4.00	6.1	2.70	3.4
December	*1.00	2.2	*1.50	2.9	1.51	3.9	1.13	3.2	*3.00	4.6	*1.50	1.9
Year	45.99	.	51.34	. . .	39.20	. . .	34.54	. . .	65.67	. . .	79.00	. . .

Month	N. Yakima, Wash., ground pan diameter—four feet		Hermiston, Oreg., ground pan diameter—three feet		Ady, Ore., floating pan diameter—four feet		Brawley, Cal., ground pan diameter—six feet		Mammath, Cal., ground pan diameter—six feet		Granite Reef, Ariz., ground pan diameter—four feet	
	Evaporation	Per cent of yearly	Evaporation	Per cent of yearly	Evaporation	Per cent of yearly	Evaporation	Per cent of yearly	Evaporation	Per cent of yearly	Evaporation	Per cent of yearly
January	*1.75	2.6	*1.25	1.8	0.50	0.9	3.05	2.9	4.21	3.4	4.59	4.0
February	*2.50	3.7	*1.25	1.8	1.25	2.3	5.00	4.8	5.67	4.5	*4.75	4.1
March	*6.25	9.3	*3.00	4.4	3.57	6.7	8.00	7.7	8.99	7.2	*6.25	5.4
April	7.91	11.6	7.28	10.7	6.64	12.3	10.74	10.4	12.02	9.6	*9.00	7.7
May	8.36	12.3	7.89	11.6	7.15	13.4	13.79	13.3	15.52	12.4	*11.50	10.0
June	8.90	13.1	9.54	14.0	6.99	13.1	13.68	13.2	16.75	13.3	*13.50	11.7
July	10.74	15.8	12.04	17.8	8.01	15.0	14.14	13.6	18.00	14.3	*14.25	12.4
August	9.41	13.8	11.07	16.2	9.21	17.2	11.26	10.9	13.73	10.9	14.23	12.3
September	5.51	8.1	7.35	10.9	6.13	11.5	10.15	9.8	12.16	9.7	13.76	12.0
October	3.15	4.6	3.88	5.7	2.50	4.8	6.99	6.8	9.49	7.6	11.31	9.8
November	*2.00	2.9	*2.00	2.9	1.00	1.9	4.09	4.0	5.26	4.2	7.39	6.4
December	*1.50	2.2	*1.50	2.2	.50	.9	2.66	2.6	3.70	2.9	4.65	4.0
Year	67.98	.	68.05	. . .	53.45	. .	103.55	.	125.53	. . .	115.18	. . .

* Evaporation interpolated by plotting all the data available and extending the curves to cover the missing periods.

The records for Chestnut Hill Reservoir were obtained by Desmond Fitz Gerald. For the summer months they are the means of ten years of

observations, while for the winter months they are deduced from special experiments on the evaporation from snow and ice.

The data at Mount Hope Reservoir were obtained by Emil Kuichling since 1891, and are the means of two to eight years' observations. The values for these two stations as here published are taken from Turneaure and Russell's "Public Water Supplies."

The monthly percentages for the evaporation at Boston and Rochester having been derived from the mean of several years' records are of general value while those for the other stations in the table, derived from records of a year or less in length, are of smaller value since the yearly evaporation varies considerably.

Evaporation from a body of water is measured by determining the loss of water from a pan (Pl. XI, A), so placed that the contained water has as nearly as possible the same temperature and exposure as that of the water which it is intended to represent.

The seasonal differences in evaporation are illustrated by the following table, which shows the rainfall, run-off, and loss during the winter and summer months respectively in the northeastern United States. While no measurements are available showing evaporation surfaces, the table indicates that the losses during the winter months vary inversely with the latitude and therefore with the temperature.

When the temperature is below freezing for a considerable part of the time, the losses are small. During the summer or growing period, the losses are uniform and apparently have no relation to the latitude. In general the monthly loss during growing seasons in the humid sections of the country is about 3⅛ inches.

TEMPERATURE.

Temperature affects stream flow in two ways: First in the total flow, on which it acts indirectly through its effect on other climatic conditions, especially evaporation and rainfall; second, in the distribution of flow, for which it is one of the principal regulating factors, by temporarily holding back the water in the ground or in the form of snow and ice.

At the beginning of winter the formation of ice on the surface of streams, lakes, and swamps materially reduces the quantity of water available for stream flow until again released by the breaking up of the ice. For example,[a] the low-water flow of Rum River in Minnesota, during January or February, is about 70 second-feet. The river above

[a] See U. S. Geol. Survey Water Supply Paper No. 337.

the gaging station is approximately 100 miles long, its average width is about 100 feet, and its gradient is small. Ice forms over its entire surface ranging from 1½ to 2 feet in thickness. If in two months a 2-foot ice cover is formed, approximately 80,000,000 cubic feet of water will be stored as ice, an amount equal to about 15 second-feet flow, or about 21 per cent of the low-water flow of the river at the gaging section for these two months.

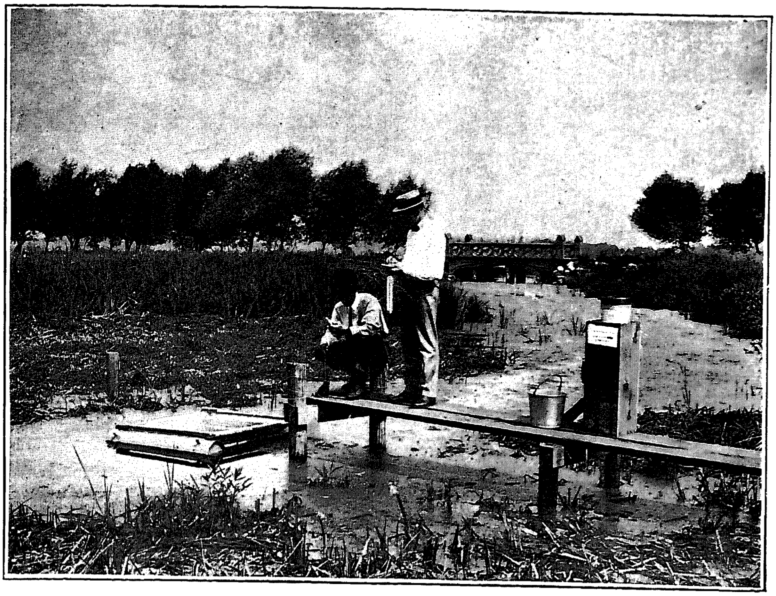
The freezing of the water also temporarily affects stream flow by the sudden increase of friction due to the ice cover, thus causing the flow at a given cross section to decrease until the slope, area of cross section and velocity have been adjusted to the new conditions. At the beginning of each cold period, therefore, stream flow will drop suddenly, but may increase to some extent later. In addition to the surface water that is held back in the form of ice, considerable quantities of ground water are frozen and the general flow of ground water is retarded, thus reducing the amount of water that reaches the streams during these periods.

Precipitation during winter usually occurs in the form of snow and therefore is available for run-off only when the temperature rises sufficiently for melting. In fact, in many sections most of the precipitation does not affect stream flow until the spring break-up. Small quantities of rain falling on snow are absorbed by it and held in storage. Though considerable melting may occur during short periods of rain or at temperatures above 32° without rain, most of the water is absorbed and held by the snow.

The magnitude of the effect of snow and ice storage varies widely with latitude and elevation and with precipitation during the winter season. Relatively few measurements of the water equivalent of such storage have been made. The table on page 157 shows the results of such measurements made in one season on the basins of small streams in the White Mountains of New Hampshire.

The importance of snow storage on the regimen of streams is illustrated by figure 37, in which the spring floods shown on the hydrographs for Kennebec River in Maine and Grand River in Colorado are caused largely, if not entirely, by the melting of snow and ice. Figure 38 illustrates in the diurnal fluctuation of stage of Kings River in California the changes in stream flow resulting from the unequal melting of snow and ice at different hours of the day in the mountains drained by that river.

Western streams in whose basins the annual precipitation is largely concentrated in the winter season are particularly dependent on snow fields and glaciers in mountainous portions of their basins for sustaining



A. PRECIPITATION AND EVAPORATION STATION, MADISON, WIS.

B. SNOW OBSERVATION STATION, WHITE MOUNTAINS, N. H.

the summer flow and therefore for the value of the streams for use for irrigation and power. Without such natural storage, an equivalent use of the streams would involve elaborate and expensive works for artificial storage which, indeed, would generally be impracticable on account of the steepness of the basins and consequent lack of feasible sites for reservoirs.

The condition of the ground when the snow cover is formed affects greatly the winter and spring run-off. If the snow falls on unfrozen ground the heat of the ground gradually melts the bottom layers of snow and the water passes into the ground. When the break-up comes the ground is in condition to absorb a part of the water, thus reducing the surface flow to streams. On the other hand, if the snow falls on frozen ground comparatively little water flows into the ground, either by melting or during the winter or the break-up. The water derived from melting snow or from rain on frozen ground flows to the drainage channels with little delay or small loss, much as it would flow from the roof of a house. Such conditions are therefore conducive to severe freshets.

The conditions of frost that may produce extreme floods also produce extremely low flows. Precipitation is stored as snow and thus prevented from reaching the streams or ground water. Part of the water already in the ground is frozen, and the only water reaching the streams is derived from that part of the ground water that is still available. In many sections of the country extremely low flows occur during the winter; in fact, in the colder parts of the country many small drainage basins are completely frozen, particularly in regions of small relief and shallow drainage channels. The following table[a] of low discharges illustrates this condition.

The rate at which water stored as snow and ice finally passes into the streams depends largely on both the temperature and its variations. Sudden and well maintained rises in temperature release the water quickly and if at the same time there is additional precipitation in the form of rain, as there often is, excessive floods may occur. If, however, rises in temperature alternate with periods of frost, the water will be released slowly and may flow off with little or no flood. Such variations also allow more water to be absorbed by the ground than a change by which the water is suddenly released and flows off rapidly. The same conditions of temperature that may produce spring freshets may therefore also produce stages of extreme low water during the summer, whereas the conditions that produce ordinary spring flows tend to well-sustained summer flows.

[a] See U. S. Geol. Survey Water Supply Paper No. 337.

Rainfall, run-off, run-off in percentage of rainfall, and loss, for the winter and the summer months, for the mean year.[a]

Station.	Winter Months, Dec. to Apr., Inclusive.				Summer Months, June, July, August.				Total yearly loss.
	Rainfall.	Run-off.	Percentage of run-off.	Loss.	Rainfall.	Run-off.	Percentage of run-off.	Loss.	
Connecticut, at Orford, N. H.	12.89	11.19	87	1.70	12.00	3.87	32	8.13	15.10
Housatonic, at Gaylordsville, Conn.	17.80	17.12	96	0.68	16.02	5.12	32	10.90	18.43
Susquehanna, at Harrisburg, Pa.	14.48	13.58	94	0.90	12.25	2.85	23	9.40	18.29
Susquehanna, at Wilkes-Barre, Pa.	14.47	16.48	114	—2.01	14.00	2.74	20	11.26	16.66
Susquehanna, at Williamsport, Pa.	15.48	14.76	95	0.72	12.87	3.30	26	9.57	17.76
Ohio, at Wheeling, W. Va.	16.23	15.16	93	1.07	12.61	3.12	25	9.49	19.02
Potomac, at Point of Rocks, Md.	14.14	9.14	65	5.00	11.80	2.44	21	9.36	22.64
Shenandoah, at Millville, W. Va.	14.38	7.72	54	6.66	12.60	2.80	22	9.80	24.69
James, at Cartersville, Va.	16.95	10.76	68	6.19	13.69	3.51	26	10.18	24.77
James, at Buchanan, Va.	15.99	10.37	65	5.62	12.87	2.97	23	9.90	24.26
North (of James) Glasgow, Va.	15.89	9.57	60	6.32	12.69	2.98	23	9.71	24.77
ittox, at Mattoax, Va.	16.87	9.89	59	6.98	14.36	3.05	21	11.31	26.50
e, at Roanoke, Va.	16.50	9.84	60	6.66	13.48	3.63	27	9.85	24.99
e, at Randolph, Va.	17.53	9.53	54	8.00	14.60	4.62	32	9.98	25.14

[a] For the number of years records see tables on pp. 160–163.

The sudden breaking of the ice cover of rivers will cause ice flows which may jam at narrow points or on riffles, and thus create temporary dams behind which large quantities of water may be stored. These temporary dams produce abnormally high stages in the pools above them, and when they break the stored water is released and causes high stages in the channel below. The effect of these dams is prolonged and increased by freezing temperatures.

The quantity and distribution of water in streams that flow from high altitudes, where snow or ice remains during all or a large part of the year, depend primarily on the temperature of the air over the snow or ice fields. As a rule such streams fluctuate in stage daily, the high stage corresponding to the period of greatest melting as shown for Kings River, Cal., by figure 38.

WIND AND HUMIDITY.

Wind and humidity affect the total flow of streams through thei effect on other climatic conditions, especially on precipitation anc evaporation.

It has been found that the movement of air adjacent to a surface o

Relation between minimum run-off of Minnesota streams and precipitation in the drainage basins in the summer of 1910 and the winters of 1911–12 and 1912–13.

River.	Station.	Drainage area, square miles.	Precipitation, April to July.				Discharge, 1910, July or August.	Precipitation, September to December.				Discharge 1912, January or February.	Precipitation, September to December.			Discharge 1913, January or February.	Flow.
			Normal.	1910	Difference.	Per cent.		Normal.	1911	Difference.	Per cent.		1912	Difference.	Per cent.		
Hudson Bay drainage basin.																	
Ottertail	Fergus Falls	1,310	13.70	6.06	− 7.64	0.44	32.6	5.92	6.28	+0.36	1.06	70	6.52	+0.60	1.10	77	Natural.
Pelican	do	433	13.70	6.06	− 7.64	.44	5.6	5.92	6.20	+ .28	1.05	2	6.57	+ .65	1.11		Do.
Wild Rice	Twin Valley	805	13.25	5.56	− 7.69	.42	21	6.59	5.32	−1.27	.81	3	6.60	+ .01	1.00	9	Controlled.
Red Lake	Thief River Falls	3,450	13.50	5.50	− 8.00	.41	236	6.50	5.10	−1.40	.78	4	6.20	− .30	.95	85	Natural.
Do	Crookston	5,320	13.08	5.46	− 7.62	.42	293	6.41	5.05	−1.36	.79	28				120	Controlled.
Clearwater	Red Lake Falls	1,310	11.18	5.43	− 5.75	.49	35.7	5.53	5.43	− .10	.98	25	6.76	+1.23	1.22		Natural.
Littlefork	Littlefork	1,720	14.35	7.90	− 6.45	.55	88.4	7.72	4.84	−2.88	.63	70	6.75	− .97	.88	58	Do.
Bigfork	Big Falls	1,320	14.23	7.49	− 6.74	.53	65.3	6.38	4.88	−1.50	.77	30	6.87	+ .49	1.08		Do.
Vermilion	Below Vermilion Lake	507	14.70	8.30	− 6.40	.56		8.20	7.30	− .90	.89	95	5.91	−2.29	.72	100	Do.
Mississippi River drainage basin.																	
Mississippi	Above Sandy River	4,510	13.97	7.94	− 6.03	.57	1,915	7.20	5.68	−1.52	.79	498	5.11	−2.09	.71	544	Controlled.
Do	Anoka	17,100	13.13	7.15	− 5.98	.54	3,390	6.28	7.26	+ .98	1.16	2,330	4.04	−2.24	.64	1,910	Do.
Do	St. Paul	35,700	13.22	7.15	− 6.07	.54	3,850	6.27	9.23	+2.96	1.47	2,700	4.02	−2.25	.64	2,330	Do.
Crow Wing	Pillager	3,230	13.97	7.00	− 6.97	.50	283	6.14	7.05	+ .91	1.15	340	4.28	−1.86	.70	345	Natural.
Sauk	St. Cloud	815	13.02	6.64	− 6.38	.51	42.6	6.00	7.99	+1.99	1.32	30	3.05	−2.95	.51		Controlled.
Elk	Big Lake	615	13.98	6.70	− 7.28	.48		6.53	8.89	+2.36	1.36	50	3.48	−3.05	.53	65	Natural.
Crow	Rockford	2,520	12.85	6.66	− 6.19	.52	58.5	7.21	11.04	+3.83	1.53	40	2.80	−4.41	.39	45	Do.
Rum	Cambridge	1,160	13.98	6.70	− 7.28	.48	84.2	6.53	8.89	+2.36	1.35	70	3.48	−3.05	.53	58	Do.
Minnesota	Odessa	1,560	12.79	5.49	− 7.30	.43	16.5	5.25	9.67	+4.42	1.84	4	3.16	−2.09	.60	3	Do.
Do	Montevideo	6,300	13.05	8.35	− 4.70	.64	72.5	5.54	9.94	+4.40	1.79	15	2.92	−2.62	.53	76	Do.
Do	Mankato	14,600	13.51	7.71	− 5.80	.56	253	5.66	10.56	+4.90	1.87	250	3.54	−2.12	.63	131	Do.
Chippewa	Watson	1,940	13.51	8.01	− 5.50	.59	12	5.44	8.69	+3.25	1.59	5	2.76	−2.68	.51	2	Do.
Cottonwood	New Ulm	1,190	13.93	9.21	− 4.72	.66	9.3	5.86	12.63	+6.77	2.16	15	3.92	−1.94	.67	3	Do.
Kettle	Sandstone	825	14.50	7.00	− 7.50	.48	71.1	9.27	10.09	+ .82	1.09	90	4.40	−4.87	.47	40	Do.
Snake	Mora	422	14.04	6.34	− 7.70	.45	37.5	8.82	10.50	+1.68	1.19	30	3.61	−5.21	.41	23	Do.
Cannon	Welch	1,290	14.06	5.06	− 9.00	.36	124	8.04	14.65	+6.61	1.83	310	5.46	−2.58	.68	100	Controlled.
Zumbro	Zumbro Falls	1,120	14.61	4.54	−10.07	.31	157	7.68	15.32	+7.64	2.00	242	5.82	−1.86	.76	145	Natural.
Zumbro, South Branch	do	821						7.68	15.32	+7.64	2.00	165	5.82	−1.86	.76	80	Do.
Root	Houston	1,560	15.44	4.65	−10.79	.30	322	8.29	15.88	+7.59	1.92	400	7.12	−1.17	.86	270	Do.
Root, North Branch	Lanesboro	647	15.44	4.65	−10.79	.30	134	8.29	15.88	+7.59	1.92	150	7.12	−1.17	.86	100	Do.
Cedar	Austin	425	15.43	5.25	−10.18	.34	49.5	7.87	15.84	+7.97	2.01	45	7.06	− .81	.90	33	Controlled.

evaporation is one of the principal factors that determines the rate of evaporation. The distribution of precipitation is also determined to a considerable extent by the direction of prevailing winds.

Wind affects the distribution of flow of streams that are fed by lakes and ponds. The quantity of water passing into the stream will vary with the intensity and direction of the wind, which may blow the water toward or away from the outlet. Wind generally has little effect on the velocity of water in a stream.

VEGETATION.

In considering vegetation with respect to its effect on stream flow, three types of vegetal cover should be recognized: 1. Forest, either virgin or sizable second growth. 2. Brush of either perennial or annual plants, including small second growth. 3. Agricultural crops, native grasses and other small wild plants. The effects of these three types are similar and their influence extends both to the total run-off and the regimen of flow.

Vegetation affects the total flow of a stream principally through its effect on evaporation and in a less degree by absorption in growing plants. In general, it decreases the total run-off. As shown in the tables on pp. 160–163, the monthly loss to the flow of a stream during the growing season amounts to about 3½ inches in depth over the drainage area. Though many conditions contribute to this loss, vegetation is believed to be an important factor.

Vegetation affects the regimen of a stream by delaying run-off in various ways. It increases ground storage because of the greater receptivity of a soil loosened and opened by roots and a surface covered with fallen leaves and litter. The roots and cover retard the flow of water over the surface and thereby promote the absorption of the water by the soil. The effect of forests on ground storage is therefore least on open, sandy soils, which would readily absorb water under all conditions, and greatest on heavy, compact soils and clays, which do not take up water easily.

Vegetal cover is an important factor affecting the rate of melting of snow. In forest areas, especially in evergreens, the melting of snow is delayed and the delay may increase or decrease flood flows in accordance with other circumstances. If the winter is followed by a period in which temperatures are moderate and rainfall is small, the accumulated snow will tend to maintain the stream flow during the spring and summer months. On the other hand, if high temperatures and heavy rains prevail, the accumulated snows will be rapidly carried into the

streams and may cause disastrous floods. In fact, many serious floods have been due to this cause. An accumulation of snow in drifts tends to delay melting and prolongs spring run-off. Vegetation may be an important factor in the formation or prevention of drifts.

The difference between evergreen and deciduous forests or between large and small trees in their effects on ground storage are probably small. Young trees and bushes, or even cultivated crops and small wild growth, may produce approximately the same effects in this respect as virgin forests. In their effects on temperature and resultant evaporation, and especially on snow storage, forests differ greatly. Dense evergreen growths hold the snow longer than open or deciduous growths.

It is doubtful whether vegetation commonly affects the flow of streams through its effect on rainfall. Although forested areas have ample rainfall, it is believed that the forests exist because of this rainfall, rather than the converse.

None of the many attempts that have been made to study the magnitude of the effects of forest on stream-flow has given satisfactory results. Such conclusions as are available on the subject are based on general observations rather than on measured difference in stream-flow. In order that observations may furnish decisive evidence on this subject, it is necessary that they be made under similar conditions of climate, geology and topography, and that the only differences shall pertain to forestation. Such conditions are not readily found or produced. The numerous published discussions in regard to the effect of forest on stream-flow are as a rule argumentative in character and fail in logical presentation of the subject. Indeed the possible effects of other influencing factors have seldom if ever been given adequate consideration. The relative effect of vegetation will depend largely on the magnitude of the other factors that affect stream-flow and many irregularities in the run-off, attributed to this cause, are probably due to other conditions.

Vegetation has a marked influence on streams and their character through its effect on erosion in the basin. Under similar conditions of soil the amount of silt carried in streams varies inversely with the extent and type of the vegetal cover. Streams carrying large quantities of silt, usually characterized by shifting channels, are less readily available for use, and are liable to obstruction and damage from floods.

TOPOGRAPHY.

The direct effects of topography on stream flow appear in the distribution or regimen of flow. Indirectly by its effect on climate, topography

affects the total run-off. The topographic factors comprise (a) elevation of drainage basin; (b) slope of drainage basin; (c) slope of bed of stream; (d) shape of drainage basin; (e) direction of axis; and (f) lake and swamp areas.

Mountain ranges, by intercepting the moisture and the clouds in the atmosphere, cause heavy precipitation on their summits and slopes, thereby decreasing the moisture available for precipitation beyond the range. The snow stored in mountainous areas is the principal source of the water of many western streams. The magnitude of the effect of snow storage on the summer flow will depend on the elevation at which the snow accumulates, and on the position of drifts with respect to slopes. For the same altitude the storage will produce the most lasting effect where the drifts accumulate on the north or other slopes that are little exposed to the sun. Drifts on southern slopes or at low altitudes melt more rapidly and have little effect in sustaining summer flow.

The capacity for ground storage from which water may reach the streams depends on the volume of earth above the stream bed and is determined by the topography of the basin. A low, swampy area in which stream channels are shallow has a small ground-storage capacity and, consequently, little reserve for maintaining flow during periods of drought. Precipitation on such an area finds its way to the streams slowly because of small slopes and low velocities, and flood stages occur only after long periods of heavy precipitation. These conditions also favor maximum evaporation.

The nature of the slopes of the basin and stream affects principally the distribution of run-off. Steep slopes discharge their water rapidly and as a rule store relatively small amounts of ground water. Flat and rolling areas, on the other hand, if pervious and sufficiently elevated above the stream beds to have large storage capacity, absorb much of the water that falls upon them and yield it up gradually. Rivers draining steep slopes may therefore be "flashy" in character, whereas those that drain flat and rolling areas fluctuate less rapidly.

The shape and size of a drainage basin affects decidedly the streams transversing it. The flow from basins which are so large that they have considerable range in latitude may be materially affected owing to the difference in the times at which the snows are melted. Rivers flowing southward may discharge the snow water without serious freshet, for the melting of the snows beginning in the southern part extends gradually toward the north. Rivers flowing northward, on the other hand, are more liable to freshets from the accumulation of water as the higher temperatures advance north, and to ice jams and consequent freshets

from backwater therefrom, because of the greater thickness and strength of the ice farther north. Rivers flowing east and west are more likely to have the same conditions of temperature throughout the whole basin and consequently an accumulation of water resulting from the melting of snow simultaneously in all tributaries. The shape of the basin, whether long or palmate, affects in a similar way the accumulation of freshet waters. The magnitude of these effects will not be large, except on long rivers because the time interval involved in changes in temperature throughout the basin will not ordinarily be great.

The flow of large rivers that are fed by small streams whose combined drainage areas are large is generally more regular than that of streams draining small areas, as the diversity of climate, topographic, geologic and other conditions that affect flow averages the run-off resulting from the various tributary areas.

The shape of the basin and the direction of its axis relative to the direction of motion of prevailing storms may be important factors in determining the magnitude of freshets. The progress of storms in the same direction as the current of a stream rather than against it tends to augment the freshet discharges.

Lakes and ponds affect stream-flow (1) by decreasing the total annual run-off, on account of the evaporation from their surfaces, and (2) by equalizing flow and making the discharge more regular. They are also important in connection with the use of water on account of their availability for artificial storage. Swamp areas affect the flow of streams, especially as regards their regimen, in much the same way as lakes and ponds.

GEOLOGY.

Aside from its effect on topography, which is a direct result of it, geology has an important influence on stream-flow in two ways; first, in the nature and depth of the soil and, second, in the dip of the strata. It affects both the total run-off and the regimen.

Sandy and other porous soils afford maximum capacity for ground storage. Rain falling on such porous soils passes into the ground readily, losing a minimum quantity by evaporation, and is retained there temporarily to be gradually released to springs and rivers. An example of this effect is shown by Loup River, which drains a sandy basin in Nebraska, and the Republican, which drains a less porous basin in the same State. The tables on p. 175 give the flow of these two streams for a typical year. Withlacoochee, Silver Spring, and other

rivers in Florida are representative of streams draining sandy areas and having small fluctuations.

Similarly, porous lava absorbs large quantities of water and later gives it up in springs or streams of well-sustained flow. Snake River in Idaho below Thousand Springs and Deschutes River in Oregon (see table on p. 175) are streams of this character. Streams in both sandy and lava areas derive their water mainly from springs and other forms of ground water flow and have comparatively few tributaries.

Basins of bare rock, on the other hand, shed quickly the most of the water that falls upon them. Between the two extremes of rock and sand are all grades of clay, silt and loam, of varying depths, affecting very decidedly the regimen of the streams that drain them.

The dip, hardness and porosity of the strata are often important on account of their effect on the courses of streams and on the concentration of fall, and also on the amount of water absorbed by the ground to appear elsewhere as springs in the same or another drainage basin, or to be lost permanently to the ground, unless it is brought to the surface again by means of some deep well.

GEOGRAPHIC LOCATION.

Geographic location influences stream flow principally through its effect on climate and vegetation. The climatic conditions already described, which, in turn, determine the amount of vegetation, depend largely on geographic location, and their effect on stream flow has been discussed (pp. 155 to 171).

THE WORKS OF MAN.

In the industrial development of the water resources of the country, it is necessary to construct dams, dikes, diversion channels, and other works which have a large direct effect on the flow of streams. The operations in connection with agriculture, the construction of city improvements, and the development of transportation also indirectly affect stream flow.

Irrigation affects the total flow by creating additional water surfaces for evaporation and by losses in irrigation as only a part of the water diverted to the land returns to the stream. It affects the regimen by the regulating works, which may include storage reservoirs, by diversion during the irrigating season, and by promoting seepage or return waters that may appreciably augment the flow during the non-irrigating season.

The use of a stream for power affects the total flow only by such in-

Monthly discharge of Loup River at Columbus, Nebr., for 1906.

[Drainage area, 13,500 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.		Rainfall depth in inches.
	Maximum.	Minimum.	Mean.		Sec.-ft. per sq. mile.	Depth in inches.	
April (8-30)	16,100	2,500	5,580	255,000	0.413	0.35	5.47
May	25,000	1,500	4 590	282,000	.340	.39	2.89
June	4,800	1,900	2,890	172,000	.214	.24	2.06
July	7,900	1,700	2,780	171,000	.206	.24	2.74
August	11,600	1,000	3,120	192,000	.231	.27	3.52
September	7,400	1,100	2,600	155,000	.193	.22	2.94
October	7,100	1,900	3,280	202,000	.243	.28	3.12
November (27 days)	4,900	2,000	3,090	165,000	.229	.23	.93
The period	25,000	1,000	3,490	1,590,000	.259	2.22	23.67*

* Total year, 27.03.

Monthly discharge of Republican River near Bostwick, Nebr., for 1906.

[Drainage area, 23,300 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.		Rainfall depth in inches.
	Maximum.	Minimum.	Mean.		Sec.-ft. per sq. mile.	Depth in inches.	
April (7-30)	1,690	480	673	32,000	0.029	0.03	[illegible]
May	5,130	750	1,610	99,000	.069	.08	1.87
June	750	260	466	27,700	.020	.02	2.02
July (8-31)	2,140	210	736	35,000	.032	.03	2.99
August	1,590	260	553	34,000	.024	.03	2.89
September	460	150	253	15,100	.011	.01	2.04
October	790	170	371	22,100	.016	.02	2.53
November	630	525	610	36,300	.026	.03	.67
The period	5,130	150	659	301,000	.028	.25	19.93*

* Total year, 23.47.

Monthly discharge of Deschutes River at Benham Falls, near Bend, Oreg., for the year ending Sept. 30, 1911.

Month.	Discharge in second-feet.			Run-off (total in acre-feet).
	Maximum.	Minimum.	Mean.	
October	1,540	1,420	1,470	90,400
November	1,600	1,420	1,490	88,700
December	1,780	1,420	1,560	95,900
January	1,540	1,180	1,420	87,300
February	1,420	1,240	1,360	75,500
March	1,540	1,240	1,400	86,100
April	1,670	1,480	1,570	93,400
May	1,740	1,600	1,670	103,000
June	1,740	1,600	1,710	102,000
July	1,600	1,420	1,500	92,200
August	1,420	1,360	1,380	84,800
September	1,480	1,360	1,390	82,700
The year	1,780	1,180	1,490	1,080,000

crease in evaporation as may result from increased water surfaces. Its effects on the regimen will vary with the amount of storage utilized.

By drainage, both surface and subsurface, the water from precipitation may be conducted more quickly to the streams, thus tending under some conditions to increase flood discharge and to decrease low-water discharge. It also diminishes the effect of evaporation if swamp areas are drained, thereby increasing the total flow. The effect of drainage on stream-flow has often been underestimated or overlooked.

Diversion and storage works constructed for city water supplies and other purposes produce the same effect as that described for irrigation, power and drainage. The general effect of storage is to reduce the total flow by increasing evaporation and to modify the regimen generally by making the discharge more nearly uniform at all seasons. Diversion for use always diminishes the flow, the amount depending on the nature of the use and varying from the small losses in power plants to the large losses in irrigation. Water diverted outside of the basin is of course a complete loss to that basin and a net gain to the basin to which the diversion is made. Works in and along the river channel, such as the building of dikes to prevent overflow and to improve navigation, the construction of wharves, bridge abutments, and piers, contract the natural channel and tend to increase the stage. These works have been important factors in increasing the damages resulting from many floods.

In considering effects on stream flow little consideration has been given to the indirect effects of the works of man. The preparation of virgin land for agriculture may necessitate the clearing of timber and other vegetation, leveling of the surface, and drainage, both surface and subsurface. Such preparation and the agricultural preparations that follow influence to a greater or less extent the run-off from the areas affected. In general, they affect the regimen. Conditions for more rapid run-off are created and greater fluctuations in discharge are promoted.

The construction of city improvements and the development of transportation have an effect on stream flow similar to that resulting from agricultural operations. Conditions that promote rapid run-off, as discussed, tend to increase the total annual run-off. The works incident to the development of a country have a further effect on streams in promoting erosion, thereby increasing the quantity of silt to be carried, which may have an important effect on the characteristics of the stream.

FLOODS.

The normal capacity of the channel of a stream is developed by natural processes and is adequate to carry the ordinary flow. Overflow of banks, or floods, will occur therefore only as a result of extraordinary conditions that bring to the channel a quantity of water in excess of its capacity. Such conditions may be (1) excessive rainfall, (2) rapid melting of accumulated snow, (3) failure of reservoirs, (4) forming and breaking of jams of logs, ice or other debris, (5) winds, or (6) tidal waves.

The characteristics of a flood, its stage and duration, may be affected favorably or unfavorably by many of the works of man. Chief among those that have an unfavorable effect are bridge piers, abutments, embankments, buildings, or other structures that constrict the channel and decrease its capacity, and levees or other protective works built for holding the water in a definite channel but which when breached release the water and permit it to spread over the adjacent lowlands. In similar manner, any structure or development, such as drainage channels, either surface or subsurface, highways, and improvements in stream channels that will shorten the time required for rain water to reach the streams, tends to increase floods, whereas structures or developments that will increase such time tend to decrease floods.

All of the many conditions affecting stream flow, described in this chapter, affect also the stage and duration of floods. Chief among these is, of course, rainfall, and particularly its intensity. Cloud-bursts are especially disastrous on small basins, and, in fact, many of the most destructive floods have occurred on small streams after intense precipitation that produced an excessive stage of short duration.

The flood[a] of August 3, 1915, on Mill Creek, at Erie, Pa., was of this character. Mill Creek drains an area of 12.9 square miles, of such slopes and strata as to be favorable to high run-off. After a month, in which the precipitation was nearly 2 inches above normal, 5.77 inches of rain fell in 13 hours, producing a maximum discharge in the creek of about 11,000 second-feet, or at the rate of nearly 1,000 second-feet per square mile.

Another flood of this character occurred on January 1, 1910, in Meadow Valley Wash, Nevada.[b] This flood, resulting from a cloudburst, washed out about 80 miles of the San Pedro, Los Angeles & Salt Lake R. R., and caused the railroad to suspend operations for several

[a] See Engineering News, Vol. 74, pp. 327 and 937, 1915.

[b] See Engineering News, Vol. 63, p. 83, 1910.

months. On a stream having a larger channel capacity, such quantities of water would have passed unnoticed.

Floods on large rivers are generally caused either by (1) long continued rainfall over a large area—perhaps at no point reaching great intensity—or (2) melting snows over a large area. These conditions will be aggravated if high temperatures or rainfall occur when there is considerable accumulated snow or if the surface of the ground is frozen. Flood stages under such conditions may be modified by many of the factors affecting stream flow already mentioned, of which storage capacity in the ground is among the most important. This factor may, however, be entirely eliminated by saturation of the ground or by an impervious frozen surface. The Ohio River flood of March, 1913,[a] is a good example of floods of this character.

Floods caused by the failure of dams or by the breaking of ice gorges are often disastrous, on either large or small streams. Such floods usually occur as flood waves of great height and short duration, and as they may come without much warning are sometimes accompanied by the loss of many lives in the valley below. The Johnstown flood[b] of May-June, 1889, and the Susquehanna River flood[c] of March, 1904, illustrate floods of this type. The Johnstown flood was due to excessive precipitation that caused the overtopping and failure of an earth dam resulting in a flood wave and great damage in Conemaugh Valley, especially in the city of Johnstown, Pa. The same storm also produced a general flood in adjacent drainage basins, which reached record stages at many points. The Susquehanna flood was caused by the forming and breaking of ice jams at various places in the main river and its tributaries.

As a rule, the damages resulting from a flood depend on the stage reached and will be largely independent of the duration of the stage. The many methods that have been proposed for controlling or preventing floods may be reduced to two groups: (1) Those for holding back the water by reservoirs or other means. (2) Those for increasing channel capacity by levees or other structures. Either or both of these methods may be used, according to conditions on the stream to be regulated.

Works for preventing floods or alleviating conditions produced by floods may cause damages at other localities not formerly subject to floods. Plans for the improvement of such conditions should be so comprehensive as to include the consideration of all stretches of the

[a] See U. S. Geol. Survey, Water-Supply Paper No. 334.

[b] See Engineering News, vols. 21 and 22, 1889.

[c] See U. S. Geol. Survey, Water-Supply Paper No. 109.

stream that may be affected, either directly or indirectly, by the proposed works.

The following table, abstracted from Emil Kuichling's discussion of "Flood Flows," contained in Transactions American Society Civil Engineers, Vol. LXXVII, shows flood discharges in many streams.

Extreme flood discharges.

Stream and locality.	Drainage area in square miles.	Maximum discharge in second-feet per square mile.	Date of flood.	Number of years observed.	Authority.
Mississippi River, at St. Louis, Mo.	702,380	1.28	June, 1883	6 (1880-85)	(16)
Colorado River, at Yuma, Ariz.	225,000	0.67	June, 1909	11 (1902-12)	(3)
Ohio River, at Paducah, Ky.	205,750	7.00	Feb., 1884	6 (1880-85)	(16)
Mississippi River, at Grafton Ill.	171,570	2.10	June, 1883	6 (1880-85)	(16)
Platte River, near Columbus, Nebr.	56,900	0.83	May 15, 1905	12 (1895-06)	(8)
Mississippi River, at Prescott, Wis.	44,070	2.50	April, 1881	6 (1880-85)	(16)
Red River, at Grand Forks, N. D.	25,000	1.70	April, 1897	31 (1882-12)	(14)
Susquehanna River, at Harrisburg, Pa.	24,030	30.6	June, 1889	41 (1865-05)	(1)
Ohio River, at Wheeling, W. Va.	23,800	20.8	Feb. 7, 1884	22 (1884-05)	(1)
Republican River, at Bostwick, Nebr.	22,300	1.10	July 4, 1905	11 [illegible]	[illegible]
Tennessee River, at Chattanooga, Tenn.	21,382	34.37	Mar. 11, [illegible]		
Alabama River, at Selma, Ala.	15,400	9.5	Jan. 19, 1892		
Loup River, at Columbus, Nebr.	13,540	5.17	June 6, 1896		
Sacramento River, at Red Bluff, Cal.	10,400	24.42	Feb., 1909		
Potomac River, at Point of Rocks, Md.	9,654	48.9	June 2, 1889	18 (1889-06)	
Penobscot River, at Bangor, Me.	7,700	14.94	April 10, 1901	35 (1875-09)	(7)
Savannah River, at Augusta, Ga.	7,500	40.00	Sept. 11, 1888	66 (1840-05)	(1)
Delaware River, at Lambertville, N. J.	6,855	37.14	Jan. 8, 1841	120 (1786-1905)	(1)
Hudson River, at Mechanicsville, N. Y.	4,500	26.67	Mar. 28, 1913	26 (1888-13)	(21)
New River, at Radford, Va.	2,725	63.78 63.37	Oct., 1900 May, 1901	13 (1898-10)	(13)
Wisconsin River, at Merrill, Wis.	2,630	8.02	Sept. 16, 1903	4 (1902-05)	(9)
American River, at Fair Oaks, Cal.	1,910	55.00	Mar., 1907	9 (1904-12)	(5)
S. Fork Shenandoah River, near Front Royal, Va.	1,570	48.92	Mar., 1902	8 (1899-06)	(10)
Catawba River, at Catawba, N. C.	1,535	61.89	May 23, 1901	10 (1896-05)	(19)
Chagres River, at Gatun, Panama	1,320	93.9	Dec. 28, 1909	20 (1894-13)	(18)
Yuba River, near Smartsville, Cal.	1,220	90.91	Jan., 1909	10 (1903-12)	(5)
Scioto River, at Columbus, O.	1,047	80.82	Mar. 25, 1913	Indefinite	(22)
Schoharie Creek, at Fort Hunter, N. Y.	900.0	55.11	Mar. 21, 1901	16 (1898-13)	(21)
Raritan River, at Bound Brook, N. J.	806	64.52	Sept. 24, 1882	96 (1810-05)	(1)
Chagres River, at Bohio, Panama	779	115.5	Dec. 27, 1909	20 (1894-13)	(18)
Little Tennessee River, at Judson, N. C.	675	85.3	Dec., 1901	15 (1896-10)	(12)
Santa Catarina River, at Monterey, Mex.	544	590.00	Aug. 27, 1909	Indefinite	(20)
Chagres River, at Alhajuela, Panama	427	398.1	Dec. 26, 1909	20 (1894-13)	(18)
Calaveras River, at Jenny Lind, Cal.	395	176.20	Jan., 1911	6 (1907-12)	(4)
W. Canada Creek, at Hinckley, N. Y.	372	104.57	April 21, 1869	45 (1869-13)	(21)
Bear River, at Van Trent, Cal.	263	98.10	Mar., 1907	9 (1904-12)	(5)
Esopus Creek, at Olive Bridge, N. Y.	239	64.39	April 26, 1910	7 (1906-12)	(6)
Catskill Creek, at S. Cairo, N. Y.	210	100.00	Spring, 1901		(15)
Rio Mora, below Mora, N. Mex.	159	139.70	Sept. 29, 1904	. . .	(2)
Devil's Creek, near Viele, Iowa	143	1,300.0	June 10, 1905	. . .	(1)
Putah Creek, near Guenoc, Cal.	91	198.90	Mar., 1904	3 (1904-06)	(5)
Elkhorn Creek, at Keystone, W. Va.	44.0	1,363.0	June 22, 1901	Indefinite	(19)
Cane Creek, at Bakersville, N. C.	22.0	1,341	May 20, 1901	Indefinite	(19)
Willow Creek, near Heppner, Ore.	20.0	1,800	June 14, 1903	Indefinite	(11)
Mill Brook, at Sherburne, N. Y.	5.0	262.0	Sept. 4, 1905	Indefinite	(1)
Estanzuela River, near Monterey, Mex.	3.50	825.0	Aug. 28, 1909	Indefinite	(20)
Cherryvale Creek, at Cherryvale, Kans.	2.00	930.0	. . .	Indefinite	(17)
Beacon Brook, near Fishkill, N. Y.	0.25	3,200.00	July 14, 1897	18 (1896-13)	(15)

Authorities: Water-Supply Papers of the United States Geological Survey, as follows:

(1) No. 162; (2) No. 147; (3) No. 300; (4) No. 299; (5) No. 298; (6) No. 281; (7) No. 279; (8) No. 230; (9) No. 156; (10) No. 192; (11) No. 96; (12) No. 75; (13) Virginia Geol. Survey, Bulletin III, Hydrography; (14) Minnesota State Drainage Com. Report for 1912; (15) Report of New York State Engineer for 1902; (16) Report on Reservoirs in Wyoming and Colorado, 1898; (17) Trans. Am. Soc. C. E., Vol. LIV, p. 200; (18) Trans. Am. Soc. C. E., Vol. LXXVI, p. 871; (19) Engineering News, Vol. 48, p. 104; (20) Engineering News, Vol. 62, p. 315; (21) Engineering Record, Vol. 67, p. 399; (22) Engineering Record, Vol. 67, pp. 444 and 592.

A general discussion of destructive floods in the United States may be found in U. S. Geol. Survey Water-Supply Papers Nos. 96, 147, 162, and 334. No. 162 contains an index to flood literature. Many references to flood literature are also given in Appendix 7 of the Report of Flood Commission, Pittsburgh, Pa.

LOW WATER.

Records of minimum discharge are important in connection with all uses of water where storage is not available. The minimum flow of a stream ordinarily depends on rainfall and ground storage. Available ground water and lake storage, in the absence of precipitation, is frequently the controlling factor in determining low-water flow. In many sections the low water occurs during the winter months, as at that time the precipitation may be in the form of snow and is not available to replenish the streams until spring. At such time, also, the frost in the ground and the ice on the surface of the streams and lakes holds back water that otherwise would be discharged. The table on p. 169 illustrates the differences in low-water flow in summer and winter and in streams with and without lake storage. The effect of frost on low-water flow is greatest in a relatively flat country where drainage channels are shallow and much of the ground water may be frozen. Lake storage, either natural or artificial, is an important factor in maintaining the low-water flow, as illustrated in the table on p. 169, by Minnesota River at Mankato and the Mississippi at Anoka, and other streams.

Streams that drain areas that afford conditions favorable for rapid run-off are subject to extremely low as well as extremely high water.

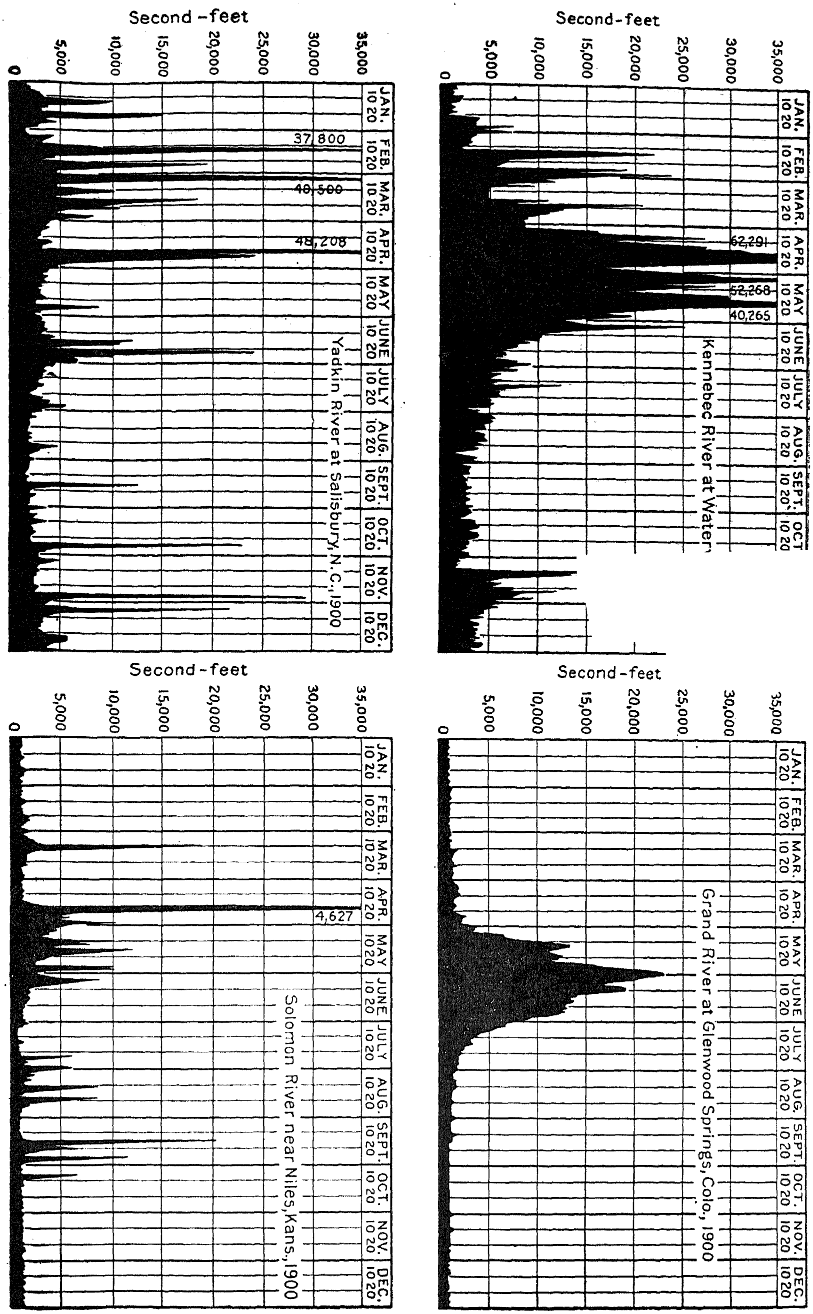


FIG. 37.—Common hydrographs of typical streams.

Authorities: Water-Supply Papers of the United States Geological Survey, as follows:

(1) No. 162; (2) No. 147; (3) No. 300; (4) No. 299; (5) No. 298; (6) No. 281; (7) No. 279; (8) No. 230; (9) No. 156; (10) No. 192; (11) No. 96; (12) No. 75; (13) Virginia Geol. Survey, Bulletin III, Hydrography; (14) Minnesota State Drainage Com. Report for 1912; (15) Report of New York State Engineer for 1902; (16) Report on Reservoirs in Wyoming and Colorado, 1898; (17) Trans. Am. Soc. C. E., Vol. LIV, p. 200; (18) Trans. Am. Soc. C. E., Vol. LXXVI, p. 871; (19) Engineering News, Vol. 48, p. 104; (20) Engineering News, Vol. 62, p. 315; (21) Engineering Record, Vol. 67, p. 399; (22) Engineering Record, Vol. 67, pp. 444 and 592.

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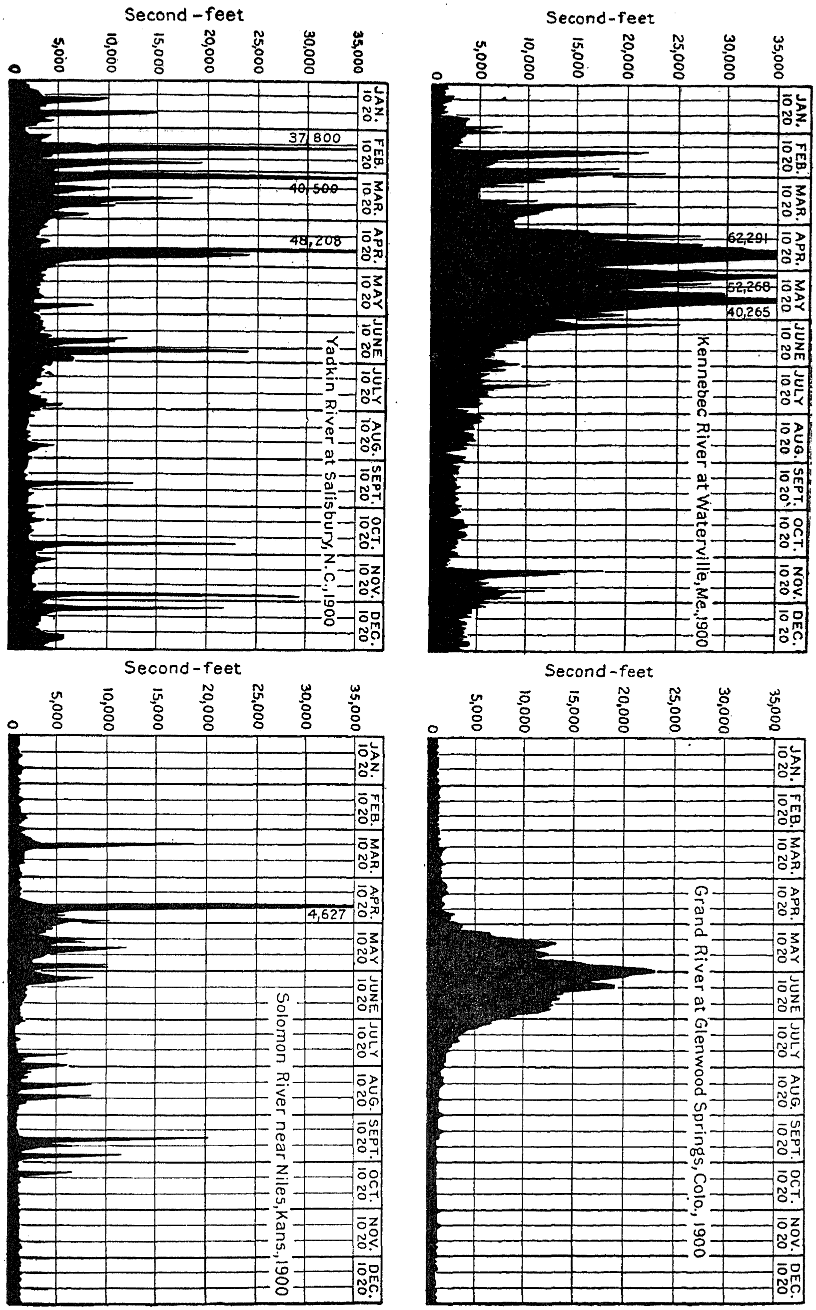

FIG. 37.—Common hydrographs of typical streams.

TYPES OF STREAMS.

Streams may be grouped, in accordance with their regimen, into four classes. The characteristics of each class depend on the climate and correspond to various geographic locations, as shown in figure 37, which gives hydrographs of typical streams.

Streams in the northeastern part of the United States are typified by Kennebec River, Maine. Their low-water flow generally occurs during the summer (growing) and winter (frozen) months, and is broken only by occasional rises caused by heavy rains; their high waters occurring during the spring months are caused by rains and melting snows. Occasional high waters occur during periods of excessive rain in the autumn and of high temperature in the winter.

Streams in the western part of the United States, draining mountainous areas and fed by melting snows, have pronounced periods of high and low water. High water usually begins in April and continues until July, and is caused by melting snow and ice. The high water is followed by gradual decrease in stage until the flood period of the next year, though occasional minor rises result from local rains. Grand River, Colorado, is typical of these streams.

Streams in the southeastern part of the United States, of which Yadkin River, North Carolina, is typical, have no defined periods of high or low water. High waters may occur at any time, depending on precipitation, and are of short duration.

Streams in the arid west, where the rainfall is usually insufficient to satisfy evaporation and other losses, of which Solomon River, Kansas, is typical, derive their flow from occasional heavy rains that may occur at any season.

These classes are intended to illustrate only general characteristics and are subject to many minor variations.

Streams may be further classified according to their daily fluctuations, as shown by the graphs made by water-stage recorders, figure 38.

Unregulated streams where conditions are not favorable for rapid run-off, rise and fall slowly with no sharp changes in stage. Sacandaga River, New York, is a stream of this type.

Unregulated streams draining area in which conditions favor rapid run-off show sharp changes in stage which may follow each other in rapid succession. An illustration is afforded by Occoquan Creek, Virginia.

Snow-fed streams, such as Kings River, California, have a pronounced daily fluctuation, depending on temperature changes.

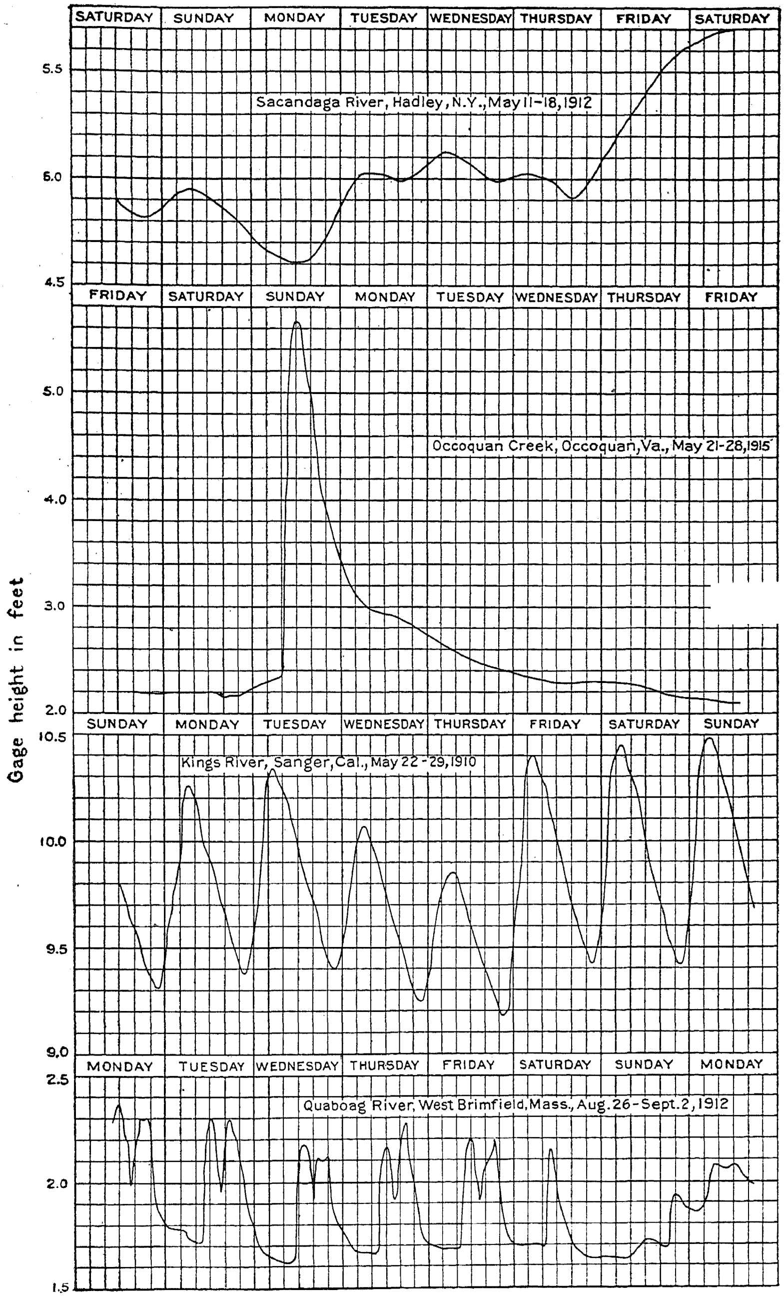

FIG. 38.—Fluctuation in stage of typical streams.

Streams used for power, illustrated by Quaboag River, Mass., have relatively large daily fluctuations below the power plant, which may be regular or irregular, depending on variations in the power load. In fact, the stage follows the load closely when the total flow of the stream is utilized.

CONCLUSIONS.

In the preceding discussion of conditions affecting stream flow it has been possible to make only general statements. The conditions are so many and varied that it is difficult to make quantitative measurements of effect for purposes of investigating any one factor. If dependable results are to be obtained, a broad study of cause and effect is necessary. Erroneous and conflicting conclusions have been reached by many investigators for the following reasons: (1) Only a part of the many factors entering the problem have been considered; (2) Cause and effect have been confused; for example, forests are often cited as a cause of rainfall, although they are probably only a result of rainfall; (3) It has been assumed that if certain causes operate under certain conditions they will operate in a similar manner under other conditions.

These fundamental errors in assumptions have led to many misinterpretations. Simple statements of conditions affecting stream flow can not, from the nature of these conditions, be made.

A systematic study of the nature of the effects of the various factors and of the changes that have taken place in stream flow during the past, requires actual records of flow extending over several decades and information in regard to the conditions and changes affecting flow. As such data are not available, any conclusions that may now be drawn are largely speculative. The testimony of the "oldest inhabitant" has generally been found by experience to be unreliable and of little value. It should be recognized also that the discharge of a stream is exceedingly variable. A comparison of yearly means with the mean for a long period of years shows large percentage variations. Therefore the effect of any factor must be large if it is to be recognized. The differences in viewpoint and the various ways of analyzing and using the data probably account largely for the differences of opinion as to the effect of the various factors on stream flow. Engineers collecting and using stream-flow data should, as occasion permits, make systematic studies of the problems involved, in order that these important conditions, so greatly affecting one of the principal branches of their profession, may in time be thoroughly understood and evaluated.

TABLES.

TABLES.

There are available a large number of tables for facilitating the computations in various hydraulic problems. It is often necessary, however, for the engineer to prepare special tables adapted to the problem in hand. Among the tables available are a number having wide application, which are given on the following pages.

These tables have been adapted from Water-Supply Papers of the U. S. Geological Survey.

In connection with the use of these tables attention is called to Barlow's tables and to Crelle's Rechentafeln. The former tables give for numbers from 1 to 10,000 the squares, cubes, square roots, cube roots, etc. The latter give products of all numbers between 1 and 1000, and can be used both for multiplication and division.

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TABLE I. Discharge in second-feet over rectangular sharp-crested weirs having complete end contractions.

$$[\text{Formula: } Q=3.33\ (l-.2H)\ H^{\frac{3}{2}}]$$

TABLE II. Discharge in second-feet per foot of crest over rectangular sharp-crested weirs without end contractions.

$$[\text{Formula: } Q=3.33\ l\ H^{\frac{3}{2}}]$$

TABLE III. Discharge in second-feet per foot of crest length for certain broad-crested weirs.

$$[\text{Formula: } Q=2.64\ l\ H^{\frac{3}{2}}]$$

TABLE IV. Discharge in second-feet per foot of crest over sharp-crested rectangular weirs without end contractions.

$$[\text{Formula: } Q=(0.405+\frac{.00984}{H})\ (1+0.55\frac{H^2}{(p+H)^2}\ lH\ \sqrt{2gH}]$$

TABLE V. Multipliers to be used in connection with Table IV to obtain the discharge over broad-crested weirs of rectangular cross-section of type *a*, Fig. 39.

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TABLE XVII. Convenient equivalents.

Table I.—Discharges in second-feet, over rectangular sharp-crested weirs having

	Head.		Length of weir.											
	Inches.	Feet.	4 in.	6 in.	9 in.	12 in.	15 in.	18 in.	24 in.	2 ft., 6 in.	3 ft.	3 ft., 6 in.	4 ft.	4 ft., 6 in.
1	1/8	0.010	0.0011	0.0017	0.0026	0.00353	......	......	......	......	......	......	......	......
2	1/4	.021	.0033	.0050	.0075	.00997	0.0125	0.0150	0.0200	0.0250	0.0300	0.0350	0.0400	0.0450
3	3/8	.031	.0060	.0091	.0137	.0183	.0229	.0275	.0367	.0459	.0551	.0643	.0735	.0821
4	1/2	.042	.0092	.0139	.0210	.0281	.0352	.0422	.0564	.0706	.0847	.0989	.113	.127
5	5/8	.052	.0128	.0194	.0293	.0392	.0491	.0590	.0788	.0986	.118	.138	.158	.178
6	3/4	.062	.0167	.0254	.0384	.0514	.0644	.0774	.103	.129	.155	.181	.207	.233
7	7/8	.073	.0209	.0318	.0482	.0646	.0810	.0974	.130	.163	.196	.229	.261	.294
8	1	.083	.0254	.0387	.0587	.0788	.0988	.119	.159	.199	.239	.279	.319	.359
9	1 1/8	.094	.0301	.0460	.0699	.0938	.118	.142	.189	.237	.285	.333	.381	.428
10	1 1/4	.104	.0349	.0536	.0816	.110	.138	.166	.222	.278	.334	.389	.445	.501
11	1 3/8	.115	.0401	.0616	.0939	.126	.159	.191	.255	.320	.385	.449	.514	.578
12	1 1/2	.125	.0454	.0699	.107	.144	.180	.217	.291	.364	.438	.511	.585	.659
13	1 5/8	.135	.0508	.0784	.120	.161	.203	.244	.327	.410	.493	.576	.659	.742
14	1 3/4	.146	.0564	.0873	.134	.180	.226	.273	.365	.458	.551	.644	.736	.829
15	1 7/8	.156	.0622	.0965	.148	.199	.251	.302	.405	.508	.611	.714	.816	.919
16	2	.167	.0680	.106	.162	.219	.276	.332	.446	.559	.672	.786	.899	1.012
17	2 1/8	.177	.0740	.115	.177	.239	.301	.364	.488	.612	.736	.860	.984	1.108
18	2 1/4	.187	.0799	.125	.193	.260	.328	.395	.531	.666	.801	.936	1.071	1.206
19	2 3/8	.198	.0862	.135	.208	.282	.355	.428	.575	.722	.868	1.015	1.161	1.308
20	2 1/2	.208	.0924	.145	.224	.303	.383	.462	.620	.778	.937	1.095	1.253	1.412
21	2 5/8	.219	......	.155	.241	.326	.411	.496	.667	.837	1.007	1.178	1.348	1.518
22	2 3/4	.229	......	.166	.257	.349	.440	.531	.714	.897	1.079	1.262	1.444	1.627
23	2 7/8	.240	......	.177	.274	.372	.469	.567	.762	.958	1.153	1.348	1.543	1.739
24	3	.250	......	.187	.291	.395	.499	.604	.812	1.020	1.228	1.436	1.644	1.852
25	3 1/8	.260	......	.198	.309	.420	.530	.641	.862	1.083	1.305	1.526	1.747	1.968
26	3 1/4	.271	......	.209	.327	.444	.561	.679	.913	1.148	1.383	1.617	1.852	2.087
27	3 3/8	.281	......	.220	.345	.469	.593	.717	.966	1.214	1.462	1.711	1.959	2.207
28	3 1/2	.292	......	.232	.363	.494	.625	.756	1.018	1.281	1.543	1.805	2.068	2.330
29	3 5/8	.302	......	......	.381	.520	.658	.796	1.072	1.349	1.625	1.902	2.178	2.455
30	3 3/4	.312	......	......	.400	.545	.691	.836	1.127	1.418	1.709	2.000	2.291	2.581
31	3 7/8	.323	......	......	.419	.572	.724	.877	1.183	1.488	1.794	2.099	2.405	2.710
32	4	.333	......	......	.438	.598	.758	.919	1.239	1.559	1.880	2.200	2.521	2.841
33	4 1/8	.344	......	......	.457	.625	.793	.961	1.296	1.632	1.967	2.303	2.638	2.974
34	4 1/4	.354	......	......	.477	.652	.828	1.003	1.354	1.705	2.056	2.407	2.758	3.109
35	4 3/8	.365	......	......	.496	.680	.863	1.046	1.413	1.779	2.146	2.512	2.879	3.245
36	4 1/2	.375	......	......	.516	.707	.899	1.090	1.472	1.854	2.237	2.619	3.001	3.384
37	4 5/8	.385	......	......	......	.735	.935	1.134	1.532	1.930	2.329	2.727	3.126	3.524
38	4 3/4	.396	......	......	......	.764	.971	1.178	1.593	2.008	2.422	2.837	3.252	3.666
39	4 7/8	.406	......	......	......	.793	1.009	1.221	1.655	2.086	2.517	2.949	3.380	3.811
40	5	.417	......	......	......	.821	1.045	1.269	1.717	2.164	2.612	3.060	3.508	3.956
41	5 1/8	.427	......	......	......	.850	1.082	1.315	1.779	2.244	2.709	3.174	3.638	4.103
42	5 1/4	.437	......	......	......	.879	1.120	1.361	1.843	2.324	2.806	3.288	3.770	4.252
43	5 3/8	.448	......	......	......	.908	1.158	1.407	1.906	2.405	2.904	3.494	3.903	4.402
44	5 1/2	.458	......	......	......	.938	1.196	1.455	1.971	2.488	3.005	3.521	4.038	4.555
45	5 5/8	.469	......	......	......	.968	1.235	1.502	2.037	2.571	3.105	3.640	4.174	4.708
46	5 3/4	.479	......	......	......	.998	1.274	1.550	2.103	3.255	3.207	3.760	4.312	4.864
47	5 7/8	.490	......	......	......	1.029	1.314	1.599	2.170	2.740	3.310	3.881	4.451	5.021
48	6	.500	......	......	......	1.060	1.354	1.649	2.237	2.826	3.415	4.003	4.592	5.181
49	6 1/8	.510	......	......	......	......	1.394	1.697	2.301	2.912	3.519	4.126	4.733	5.540
50	6 1/4	.521	......	......	......	......	1.434	1.747	2.372	2.999	3.624	4.250	4.876	5.302
51	6 3/8	.531	......	......	......	......	1.475	1.798	2.442	3.087	3.732	4.377	5.021	5.666
52	6 1/2	.542	......	......	......	......	1.516	1.848	2.511	3.175	3.839	4.503	5.166	5.830
53	6 5/8	.552	......	......	......	......	1.557	1.898	2.581	3.264	3.947	4.630	5.313	5.996
54	6 3/4	.563	......	......	......	......	1.598	1.949	2.652	3.354	4.057	4.759	5.461	6.164
55	6 7/8	.573	......	......	......	......	1.640	2.001	2.723	3.445	4.167	4.889	5.611	6.333
56	7	.583	......	......	......	......	1.682	2.053	2.795	3.536	4.278	5.020	5.762	6.504
57	7 1/8	.594	......	......	......	......	1.725	2.106	2.867	3.629	4.391	5.153	5.914	6.676
58	7 1/4	.604	......	......	......	......	1.767	2.158	2.940	3.722	4.501	5.286	6.067	6.849
59	7 3/8	.615	......	......	......	......	1.809	2.210	3.012	3.815	4.617	5.419	6.221	7.023
60	7 1/2	.625	......	......	......	......	1.851	2.263	3.085	3.908	4.731	5.554	6.376	7.199
61	7 5/8	.635	......	......	......	......	......	2.316	3.160	4.003	4.846	5.690	6.533	7.376
62	7 3/4	.646	......	......	......	......	......	2.370	3.234	4.098	4.963	5.827	6.691	7.555
63	7 7/8	.656	......	......	......	......	......	2.423	3.308	4.193	5.079	5.964	6.849	7.734
64	8	.667	......	......	......	......	......	2.477	3.384	4.290	5.196	6.103	7.009	7.915
65	8 1/4	.688	......	......	......	......	......	2.586	3.535	4.484	5.433	6.383	7.332	8.281
66	8 1/2	.708	......	......	......	......	......	2.697	3.689	4.682	5.674	6.667	7.660	8.652
67	8 3/4	.729	......	......	......	......	......	2.808	3.844	4.881	5.918	6.955	7.991	9.028
68	9	.750	......	......	......	......	......	2.919	4.001	5.082	6.164	7.245	8.327	9.408
69	9 1/2	.792	......	......	......	......	......	......	4.320	5.492	6.665	7.838	9.011	10.184
70	10	.833	......	......	......	......	......	......	4.644	5.911	7.177	8.444	9.711	10.977

complete end contractions. [*Formula* $Q = 3.33\ (l - .2H)\ H^{\frac{3}{2}}$].

Length of weir—Continued.											Addition for increase of length.		
5 ft.	6 ft.	7 ft.	8 ft.	9 ft.	10 ft.	12 ft.	14 ft.	16 ft	18 ft.	20 ft.	1 in.	1 ft.	
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	0.00029	0.00354	1
0.0500	0.0600	0.0700	0.0800	0.0900	0.100	0.120	0.140	0.160	0.180	0.200	.00083	.01001	2
.0918	.110	.129	.147	.166	.184	.221	.258	.294	.331	.368	.00153	.0184	3
.141	.170	.198	.226	.255	.283	.340	.396	.453	.510	.566	.00236	.0283	4
.198	.237	.277	.316	.356	.395	.475	.554	.633	.712	.791	.00330	.0396	5
.260	.312	.364	.416	.468	.520	.624	.728	.832	.936	1.040	.00433	.0520	6
.327	.392	.458	.524	.589	.655	.786	.917	1.048	1.179	1.310	.00546	.0656	7
.399	.479	.559	.640	.720	.800	.960	1.120	1.280	1.441	1.601	.00667	.0801	8
.476	.572	.667	.763	.859	.954	1.145	1.337	1.528	1.719	1.910	.00796	.0956	9
.557	.669	.781	.893	1.005	1.117	1.341	1.565	1.789	2.013	2.237	.00933	.112	10
.643	.772	.901	1.030	1.160	1.289	1.547	1.805	2.064	2.322	2.580	.0108	.129	11
.732	.879	1.026	1.174	1.321	1.468	1.762	2.057	2.351	2.645	2.940	.0123	.147	12
.825	.991	1.157	1.323	1.489	1.655	1.967	2.319	2.650	2.982	3.314	.0138	.166	13
.922	1.107	1.293	1.478	1.664	1.849	2.220	2.591	2.962	3.333	3.704	.0155	.185	14
1.022	1.228	1.433	1.639	1.845	2.050	2.462	2.873	3.284	3.696	4.107	.0171	.206	15
1.125	1.352	1.579	1.805	2.032	2.258	2.711	3.165	3.618	4.071	4.524	.0189	.227	16
1.232	1.480	1.728	1.976	2.225	2.473	2.969	3.465	3.962	4.458	4.954	.0207	.248	17
1.342	1.612	1.882	2.153	2.423	2.693	3.234	3.775	4.316	4.856	5.397	.0225	.270	18
1.455	1.748	2.041	2.334	2.627	2.921	3.507	4.093	4.680	5.266	5.853	.0244	.293	19
1.570	1.887	2.203	2.520	2.837	3.153	3.787	4.420	5.053	5.686	6.320	.0264	.317	20
1.689	2.029	2.370	2.711	3.051	3.392	4.074	4.755	5.436	6.118	6.799	.0284	.341	21
1.810	2.175	2.540	2.906	3.271	3.636	4.367	5.098	5.828	6.559	7.290	.0304	.365	22
1.934	2.324	2.715	3.105	3.496	3.886	4.667	5.448	6.229	7.010	7.792	.0325	.391	23
2.060	2.477	2.893	3.309	3.725	4.142	4.974	5.807	6.639	7.472	8.304	.0347	.416	24
2.190	2.632	3.075	3.517	3.960	4.402	5.287	6.172	7.058	7.943	8.828	.0369	.443	25
2.321	2.791	3.260	3.729	4.199	4.668	5.607	6.546	7.484	8.423	9.362	.0391	.469	26
2.455	2.952	3.449	3.946	4.442	4.939	5.932	6.926	7.919	8.913	9.906	.0414	.497	27
2.592	3.117	3.641	4.166	4.690	5.215	6.264	7.313	8.362	9.411	10.460	.0437	.525	28
2.731	3.284	3.837	4.390	4.943	5.495	6.601	7.707	8.813	9.918	11.024	.0461	.553	29
2.872	3.454	4.036	4.617	5.199	5.781	6.944	8.108	9.271	10.435	11.598	.0485	.582	30
3.016	3.627	4.238	4.849	5.460	6.071	7.293	8.515	9.737	10.960	12.182	.0509	.611	31
3.162	3.802	4.443	5.084	5.725	6.366	7.648	8.929	10.211	11.493	12.774	.0534	.641	32
3.310	3.981	4.652	5.323	5.994	6.665	8.007	9.350	10.692	12.034	13.376	.0559	.671	33
3.460	4.161	4.863	5.565	6.267	6.969	8.373	9.776	11.180	12.584	13.988	.0585	.702	34
3.612	4.345	5.078	5.811	6.544	7.277	8.743	10.209	11.676	13.142	14.608	.0611	.733	35
3.766	4.531	5.296	6.060	6.825	7.590	9.119	10.648	12.178	13.707	15.237	.0637	.765	36
3.922	4.719	5.516	6.313	7.110	7.906	9.500	11.093	12.687	14.281	15.874	.0664	.797	37
4.081	4.910	5.739	6.569	7.398	8.227	9.886	11.545	13.203	14.862	16.520	.0691	.829	38
4.242	5.104	5.967	6.829	7.691	8.553	10.278	12.002	13.727	15.451	17.176	.0719	.862	39
4.404	5.299	6.195	7.090	7.986	8.882	10.673	12.464	14.256	16.047	17.838	.0747	.896	40
4.568	5.497	6.427	7.356	8.285	9.215	11.074	12.933	14.791	16.650	18.509	.0775	.929	41
4.733	5.697	6.661	7.624	8.588	9.551	11.479	13.406	15.333	17.260	19.187	.0803	.964	42
4.901	5.899	6.897	7.895	8.894	9.892	11.888	13.885	15.881	17.877	19.874	.0832	.998	43
5.071	6.105	7.138	8.171	9.204	10.238	12.304	14.371	16.438	18.504	20.571	.0861	1.033	44
5.243	6.312	7.380	8.449	9.518	10.586	12.724	14.861	16.999	19.136	21.273	.0891	1.069	45
5.416	6.521	7.626	8.730	9.835	10.939	13.149	15.358	17.567	19.776	21.985	.0921	1.105	46
5.592	6.733	7.873	9.014	10.155	11.295	13.577	15.858	18.140	20.421	22.702	.0951	1.141	47
5.769	6.947	8.124	9.301	10.478	11.656	14.010	16.365	18.720	21.074	23.429	.0981	1.177	48
5.948	7.162	8.376	9.591	10.805	12.020	14.448	16.877	19.306	21.735	24.164	.101	1.214	49
6.128	7.380	8.631	9.883	11.135	12.386	14.890	17.393	19.897	22.400	24.903	.104	1.252	50
6.311	7.600	8.889	10.179	11.468	12.758	15.336	17.915	20.494	23.073	25.652	.107	1.289	51
6.494	7.822	9.149	10.477	11.804	13.132	15.787	18.442	21.097	23.752	26.407	.111	1.328	52
6.679	8.045	9.411	10.777	12.143	13.509	16.241	18.973	21.705	24.437	27.169	.114	1.366	53
6.866	8.271	9.676	11.081	12.485	13.890	16.700	19.509	22.319	25.129	27.938	.117	1.405	54
7.055	8.500	9.944	11.388	12.832	14.276	17.164	20.052	22.941	25.829	28.717	.120	1.444	55
7.245	8.729	10.213	11.696	13.180	14.663	17.631	20.598	23.565	26.532	29.499	.124	1.484	56
7.438	8.962	10.485	12.009	13.532	15.056	18.103	21.150	24.197	27.244	30.291	.127	1.524	57
7.631	9.195	10.759	12.323	13.886	15.450	18.578	21.705	24.833	27.961	31.088	.130	1.564	58
7.826	9.430	11.034	12.639	14.243	15.848	19.056	22.265	25.474	28.683	31.892	.134	1.604	59
8.022	9.667	11.312	12.958	14.603	16.249	19.539	22.830	26.121	29.412	32.703	.137	1.645	60
8.220	9.907	11.593	13.280	14.967	16.653	20.027	23.400	26.774	30.147	33.520	.141	1.687	61
8.419	10.148	11.876	13.604	15.332	17.061	20.517	23.974	27.431	30.887	34.344	.144	1.728	62
8.619	10.390	12.160	13.930	15.700	17.471	21.011	24.552	28.093	31.633	35.174	.148	1.770	63
8.821	10.634	12.447	14.259	16.072	17.884	21.510	25.135	28.760	32.385	36.010	.151	1.813	64
9.230	11.128	13.026	14.924	16.823	18.721	22.517	26.314	30.110	33.906	37.703	.158	1.898	65
9.645	11.630	13.615	15.600	17.586	19.571	23.541	27.512	31.482	35.452	39.423	.165	1.985	66
10.065	12.138	14.211	16.285	18.358	20.432	24.578	28.725	32.872	37.019	41.166	.173	2.073	67
10.490	12.653	14.815	16.978	19.141	21.304	25.630	29.956	34.282	38.607	42.933	.180	2.163	68
11.356	13.702	16.048	18.393	20.739	23.084	27.776	32.467	37.158	41.849	46.540	.196	2.346	69
12.244	14.777	17.310	19.843	22.377	24.910	29.976	35.043	40.109	45.175	50.242	.211	2.533	70

Table II.—Discharge in second-feet per foot of crest over rectangular sharp-crested weirs without end contractions.

Formula: $Q=3.33\ l\ H^{\frac{3}{2}}$.

Head H, feet.	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
0.0	0.0000	0.0033	0.0094	0.0173	0.0266	0.0372	0.0489	0.0617	0.0753	0.0899
.1	.1053	.1215	.1384	.1561	.1744	.1935	.2131	.2334	.2543	.2758
.2	.2978	.3205	.3436	.3673	.3915	.4162	.4415	.4672	.4934	.5200
.3	.5472	.5748	.6028	.6313	.6602	.6895	.7193	.7495	.7800	.8110
.4	.8424	.8742	.9084	.9390	.9719	1.0052	1.0389	1.0730	1.1074	1.1422
.5	1.1773	1.2128	1.2487	1.2849	1.3214	1.3583	1.3955	1.4330	1.4709	1.5091
.6	1.5476	1.5865	1.6257	1.6652	1.7050	1.7451	1.7855	1.8262	1.8673	1.9086
.7	1.9503	1.9922	2.0344	2.0770	2.1198	2.1629	2.2063	2.2500	2.2940	2.3382
.8	2.3828	2.4276	2.4727	2.5180	2.5637	2.6096	2.6558	2.7022	2.7490	2.7959
.9	2.8432	2.8907	2.9385	2.9865	3.0348	3.0834	3.1322	3.1813	3.2306	3.2802
1.0	3.3300	3.3801	3.4304	3.4810	3.5318	3.5828	3.6342	3.6857	3.7375	3.7895
1.1	3.8418	3.8943	3.9470	4.0000	4.0532	4.1067	4.1604	4.2143	4.2384	4.3228
1.2	4.3774	4.4322	4.4873	4.5426	4.5981	4.6538	4.7098	4.7660	4.8224	4.8790
1.3	4.9358	4.9929	5.0502	5.1077	5.1654	5.2233	5.2814	5.3398	5.3984	5.4572
1.4	5.5162	5.5754	5.6348	5.6944	5.7542	5.8143	5.8745	5.9350	5.9957	6.0565
1.5	6.1176	6.1789	6.2404	6.3020	6.3633	6.4260	6.4883	6.5508	6.6135	6.6764
1.6	6.7394	6.8027	6.8662	6.9299	6.9937	7.0578	7.1221	7.1865	7.2512	7.3160
1.7	7.3810	7.4463	7.5117	7.5773	7.6431	7.7091	7.7752	7.8416	7.9081	7.9749
1.8	8.0418	8.1689	8.1762	8.2437	8.3113	8.3792	8.4472	8.5154	8.5838	8.6524
1.9	8.7212	8.7901	8.8592	8.9285	8.9980	9.0677	9.1375	9.2075	8.2777	9.3481
2.0	9.4187	9.4894	9.5603	9.6314	9.7026	9.7741	9.8457	9.9174	9.9894	10.0620
2.1	10.1340	10.2060	10.2790	10.3520	10.4250	10.4980	10.5710	10.6450	10.7180	10.7920
2.2	10.8660	10.9400	11.0150	11.0890	11.1640	11.2390	11.3140	11.3890	11.4640	11.5100
2.3	11.6150	11.6910	11.7670	11.8430	11.9200	11.9960	12.0730	12.1500	12.2270	12.3040
2.4	12.3810	12.4590	12.5360	12.6140	12.6920	12.7700	12.8480	12.9270	13.0050	13.0840
2.5	13.1630	13.2480	13.3210	13.4010	13.4800	13.5600	12.6400	13.7200	13.8000	13.8800
2.6	13.9610	14.0410	14.1220	14.2030	14.2840	14.3650	14.4470	14.5280	14.6100	14.6920
2.7	14.7740	14.8560	14.9380	15.0210	15.1030	15.1860	15.2690	15.3520	15.4350	15.5190
2.8	15.6020	15.6860	15.7690	15.8530	15.9380	16.0220	16.1060	16.1910	16.2750	16.3600
2.9	16.4450	16.5300	16.6160	16.7010	16.7870	16.8720	16.9580	17.0440	17.1300	17.2170
3.0	17.3033	17.3899	17.4698	17.5634	17.6503	17.7376	17.8248	17.9124	18.0000	18.0876
3.1	18.1754	18.2634	18.3516	18.4399	18.5285	18.6170	18.7056	18.7945	18.8838	18.9727
3.2	19.0619	19.1515	19.2410	19.3307	19.4206	19.5105	19.6007	19.6910	19.7812	19.8718
3.3	19.9624	20.0533	20.1442	20.2354	20.3267	20.4179	20.5095	20.6011	20.6930	20.7849
3.4	20.8777	20.9690	21.0613	21.1538	21.2461	21.3390	21.4319	21.5248	21.6180	21.7113
3.5	21.8045	21.8980	21.9917	22.0856	22.1795	22.2734	22.3677	22.4618	22.5564	22.6510
3.6	22.7456	22.8405	22.9354	23.0306	23.1259	23.2211	23.3167	23.4122	23.5081	23.6040
3.7	23.6999	23.7962	23.8924	23.9887	24.0852	24.1818	24.2787	24.3756	24.4728	24.5697
3.8	24.6673	24.7645	24.8621	24.9600	25.0576	25.1555	25.2537	25.3520	25.4502	25.5488
3.9	25.6473	25.7459	25.8748	25.9437	26.0429	26.1422	26.2414	26.3410	26.4405	26.5401
4.0	26.6400	26.7399	26.8401	26.9404	27.0406	27.1412	27.2417	27.3423	27.4432	27.5411
4.1	27.6453	27.7466	27.8478	27.9494	28.0509	28.1525	28.2544	28.3563	28.4582	28.5604
4.2	28.6626	28.7652	28.8678	28.9703	29.0732	29.1761	29.2790	29.3823	29.4855	29.5890
4.3	29.6926	29.7962	29.9001	30.0040	30.1079	30.2118	30.3163	30.4205	30.5251	30.6297
4.4	30.7342	30.8391	30.9440	31.0493	31.1545	31.2597	31.3649	31.4705	31.5764	31.6820
4.5	31.7878	31.8941	32.0003	32.1065	32.2128	32.3193	32.4259	32.5324	32.6393	32.7462
4.6	32.8534	32.9607	33.0679	33.1755	33.2830	33.3906	33.4985	33.6064	33.7143	33.8225
4.7	33.9307	34.0373	34.1475	34.2560	34.3646	34.4735	34.5824	34.6913	34.8005	34.9097
4.8	35.0193	35.1288	35.2354	35.3480	35.4578	35.5677	35.6780	35.7882	35.8984	36.0086
4.9	36.1182	36.2297	36.3406	36.4515	36.5624	36.6736	36.7845	36.8961	37.0073	37.1188

Table II.—Continued.

Head *H*, feet.	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
5.0	37.2304	37.3423	37.4542	37.5661	37.6783	37.7905	37.9027	38.0153	38.1275	38.2404
5.1	38.3529	38.4658	38.5787	38.6919	38.8052	38.9184	39.0319	39.1455	39.2591	39.3726
5.2	39.4865	39.6004	39.7146	39.8288	39.9430	40.0576	40.1718	40.2867	40.4012	40.5161
5.3	40.6310	40.7462	40.5281	40.9766	41.0919	41.2074	41.3230	41.4386	41.5544	41.6703
5.4	41.7866	41.9024	42.0186	42.1352	42.2517	42.3683	42.4848	42.6017	42.7186	42.8355
5.5	42.9523	43.0700	43.1871	43.3043	43.4219	42.5394	43.6573	43.7752	43.8931	44.0109
5.6	44.1292	44.2474	44.3659	44.4845	44.6030	44.7216	44.8404	44.9593	45.0782	45.1974
5.7	45.3166	45.4359	45.5554	45.6746	45.7945	45.9140	46.0339	46.1538	46.2740	46.3939
5.8	46.5141	46.6347	46.7552	46.8757	46.9963	47.1172	47.2380	47.3589	47.4798	47.6010
5.9	47.7226	47.8438	47.9653	48.0869	48.2084	48.3303	48.4522	48.5744	48.6963	48.8185
6.0	48.9407	49.0632	49.1858	49.3083	48.4312	49.5537	49.6766	49.7999	49.9230	50.0462
6.1	50.1694	50.2930	50.4162	50.5401	50.6637	50.7875	50.9114	51.0356	51.1595	51.2837
6.2	51.4082	51.5324	51.6570	51.7818	51.9034	52.0313	52.1531	52.2813	52.4062	52.5314
6.3	52.6570	52.7822	52.9077	53.0336	53.1591	53.2850	53.4109	53.5371	53.6630	53.7892
6.4	53.9157	54.0419	54.1684	54.2950	54.4219	54.5487	54.6756	54.8025	54.9297	55.0569
6.5	55.1832	55.3116	55.4392	55.5667	55.6943	55.8221	55.9500	56.0779	56.2061	56.3343
6.6	56.4625	56.5910	56.7192	56.8478	56.9766	57.1055	57.2340	57.3233	57.4921	57.6213
6.7	57.7505	57.8801	58.0093	58.1388	58.2687	58.3982	58.5281	58.6580	58.7882	58.9180
6.8	59.0482	59.1788	59.3090	59.4428	59.5700	59.7009	59.8314	59.9623	60.0935	60.2244
6.9	60.3556	60.4868	60.6183	60.7499	60.8814	61.0129	61.1445	61.2763	61.4082	61.5404
7.0	61.6736	61.8048	61.9370	62.0692	62.2017	62.3343	62.4671	62.6000	62.7329	62.8657
7.1	62.9986	63.1318	63.2650	63.3992	63.5317	63.6653	63.7991	63.9327	64.0665	64.2004
7.2	64.3343	64.4685	64.6027	64.7369	64.8711	65.0056	65.1268	65.2750	65.4095	65.5444
7.3	65.6793	65.8145	65.9493	66.0845	66.2197	66.3552	66.4908	66.6263	66.7618	66.8977
7.4	67.0336	67.1694	67.3053	67.4415	67.5777	67.7139	67.8504	67.9869	68.1235	68.2600
7.5	68.3969	68.5337	68.6706	68.8078	68.9447	69.0818	69.2794	69.3566	69.4941	69.6316
7.6	69.7695	69.9070	70.0449	70.1827	70.3209	70.4591	70.5973	70.7355	70.8737	71.0123
7.7	71.1508	71.2896	71.4282	71.5670	71.7059	71.8451	71.9843	72.1235	72.2627	72.4743
7.8	72.5414	72.6809	72.8208	72.9603	73.1002	73.2400	73.3802	73.5201	73.6603	73.8005
7.9	73.9410	74.0815	74.2220	74.3626	74.5031	74.6439	74.7848	74.9260	75.0669	75.2081
8.0	75.3492	75.4908	75.6320	75.7735	75.9150	76.0569	76.1987	76.3406	76.4824	76.6243
8.1	76.7665	76.9087	77.0509	77.1934	77.3360	77.4784	77.6210	77.7638	77.9067	78.0496
8.2	78.1924	78.3356	78.4788	78.6220	78.7655	78.9087	79.0522	79.1957	79.3396	79.4834
8.3	79.6273	79.7711	79.9153	80.0592	80.2034	80.3479	80.4921	80.6366	80.7811	80.9260
8.4	81.0705	81.2154	81.3602	81.5054	81.6503	81.7955	81.9406	82.0862	82.2314	82.3769
8.5	82.5224	82.6682	82.8141	82.9600	83.1058	83.2517	83.3979	83.5440	83.6902	83.8367
8.6	83.9833	84.1298	84.2763	84.4228	84.5697	84.7165	84.8634	85.0106	85.1578	85.3049
8.7	85.4521	85.5996	85.7472	85.8947	86.0455	86.1897	86.3376	86.4854	86.6336	86.7815
8.8	86.9297	87.0778	87.2264	87.3745	87.5231	87.6716	87.8204	87.9689	88.1178	88.2666
8.9	88.4192	88.5647	88.7139	88.8630	89.0126	89.1617	89.3113	89.4608	89.6103	89.7602
9.0	89.9100	90.0599	90.2064	90.3599	90.5101	90.6602	90.4778	90.9609	91.1115	91.2620
9.1	91.4125	91.5633	91.7142	91.8650	92.0159	92.1671	92.3183	92.4694	92.6206	92.7721
9.2	92.9237	93.0782	93.2267	93.3785	93.5304	93.6822	93.8341	93.9863	94.1384	94.2906
9.3	94.4428	94.5950	94.7475	94.9000	95.0529	95.2054	95.3582	95.5111	95.6639	95.8171
9.4	95.9703	96.1231	96.2766	96.4298	96.5833	96.7368	96.8903	97.0442	97.1977	97.3516
9.5	97.5057	97.6596	97.8138	97.9679	98.1021	98.2763	98.4308	98.5853	98.7398	98.8943
9.6	99.0492	99.2040	99.3589	99.5141	99.6689	99.8241	99.9793	100.1344	100.2899	100.4455
9.7	100.6010	100.7565	100.9123	101.0678	101.2237	101.3799	101.5357	101.6919	101.8481	102.0042
9.8	102.1607	102.3169	102.4734	102.6299	102.7868	102.9433	103.1001	103.2570	103.4141	103.5710
9.9	103.7282	103.8853	104.0429	104.2000	104.3575	104.5121	104.6726	104.8304	104.9882	105.1461
10.0	105.3039	105.4618	105.6199	105.7781	105.9363	106.0945	106.2530	106.4115	106.5700	106.7285

NOTE:—By increasing the quantities in this table by 1 per cent the discharge by the Cippoletti formula $(Q=3.3\frac{2}{3}\ lH^{\frac{3}{2}})$ will be obtained.

Table III.—Discharge in second-feet per foot of crest length for certain broad-crested weirs.

Formula: $Q=2.64\ lH^{\frac{3}{2}}$

Head H, feet.	0	1	2	3	4	5	6	7	8	9	10
0.00	0.000	2.64	7.47	13.7	21.1	29.5	38.8	48.9	59.7	71.3	83.5
.01	.003	2.68	7.52	13.8	21.2	29.6	38.9	49.0	59.8	71.4	83.6
.02	.007	2.72	7.58	13.8	21.3	29.7	39.0	49.1	59.9	71.5	83.7
.03	.014	2.76	7.64	13.9	21.4	29.8	39.1	49.2	60.1	71.6	83.9
.04	.021	2.80	7.69	14.0	21.4	29.9	39.2	49.3	60.2	71.7	84.0
.05	.030	2.84	7.75	14.1	21.5	30.0	39.3	49.4	60.3	71.9	84.1
,06	.039	2.88	7.81	14.1	21.6	30.0	39.4	49.5	60.4	72.0	84.2
.07	.049	2.92	7.86	14.2	21.7	30.1	39.5	49.6	60.5	72.1	84.4
.08	.060	2.96	7.92	14.3	21.8	30.2	39.6	49.7	60.6	72.2	84.5
.09	.071	3.00	7.98	14.3	21.8	30.3	39.7	49.8	60.7	72.3	84.6
0.10	0.083	3.04	8.03	14.4	21.9	30.4	39.8	49.9	60.8	72.5	84.7
.11	.096	3.09	8.09	14.5	22.0	30.5	39.9	50.0	61.0	72.6	84.9
.12	.110	3.13	8.15	14.5	22.1	30.6	40.0	50.2	61.1	72.7	85.0
.13	.124	3.17	8.21	14.6	22.2	30.7	40.1	50.3	61.2	72.8	85.1
.14	.138	3.21	8.26	14.7	22.2	30.8	40.2	50.4	61.3	72.9	85.2
.15	.153	3.26	8.32	14.8	22.3	30.8	40.3	50.5	61.4	73.1	85.4
.16	.169	3.30	8.38	14.8	22.4	30.9	40.4	50.6	61.5	73.2	85.5
.17	.185	3.34	8.44	14.9	22.5	31.0	40.5	50.7	61.6	73.3	85.6
.18	.202	3.38	8.50	15.0	22.6	31.1	40.6	50.8	61.8	73.4	85.7
.19	.218	3.43	8.56	15.0	22.6	31.2	40.7	50.9	61.9	73.5	85.9
0.20	0.236	3.47	8.61	15.1	22.7	31.3	40.8	51.0	62.0	73.7	86.0
.21	.254	3.51	8.67	15.2	22.8	31.4	40.9	51.1	62.1	73.8	86.1
.22	.272	3.56	8.73	15.2	22.9	31.5	41.0	51.2	62.2	73.9	86.2
.23	.291	3.60	8.79	15.3	23.0	31.6	41.0	51.3	62.3	74.0	86.4
.24	.310	3.64	8.85	15.4	23.0	31.7	41.1	51.4	62.4	74.1	86.5
.25	.330	3.69	8.91	16.5	23.1	31.8	41.2	51.5	62.6	74.3	86.6
.26	.350	3.73	8.97	15.5	23.2	31.8	41.3	51.6	62.7	74.4	86.8
.27	.370	3.78	9.03	15.6	23.3	31.9	41.4	51.7	62.8	74.5	86.9
.28	.391	3.82	9.09	15.7	23.4	32.0	41.5	51.9	62.9	74.6	87.0
.29	.412	3.87	9.15	15.8	23.4	32.1	41.6	52.0	63.0	74.8	87.1
0.30	0.434	3.91	9.21	15.8	23.5	32.2	41.7	52.1	63.1	74.9	87.3
.31	.456	3.96	9.27	15.9	23.6	32.3	41.8	52.2	63.2	75.0	87.4
.32	.478	4.00	9.33	16.0	23.7	32.4	41.9	52.3	63.4	75.1	87.5
.33	.500	4.05	9.39	16.0	23.8	32.5	42.0	52.4	63.5	75.2	87.6
.34	.524	4.10	9.45	16.2	23.9	32.6	42.1	52.5	63.6	75.4	87.8
35	.547	4.14	9.51	16.2	24.0	32.7	42.2	52.6	63.7	75.5	87.9
.36	.570	4.19	9.57	16.3	24.1	32.8	42.3	52.7	63.8	75.6	88.0
.37	.594	4.23	9.63	16.3	24.1	32.8	42.4	52.8	63.9	75.7	88.2
.38	.618	4.28	9.69	16.4	24.2	32.9	42.5	52.9	64.0	75.8	88.3
.39	.643	4.33	9.75	16.5	24.3	33.0	42.6	53.0	64.2	76.0	88.4
0.40	0.668	4.37	9.82	16.6	24.4	33.1	42.7	53.1	64.3	76.1	88.5
.41	.693	4.42	9.88	16.6	24.4	33.2	42.8	53.2	64.4	76.2	88.7
.42	.719	4.47	9.94	16.7	24.5	33.3	42.9	53.4	64.5	76.3	88.8
.43	.744	4.51	10.0	16.8	24.6	33.4	43.0	53.5	64.6	76.4	88.9
.44	.771	4.56	10.1	16.8	24.7	33.5	43.1	53.6	64.7	76.6	89.0
.45	.797	4.61	10.1	16.9	24.8	33.6	43.2	53.7	64.8	76.7	89.2
.46	.824	4.66	10.2	17.0	24.9	33.7	43.3	53.8	65.0	76.8	89.3
.47	.851	4.70	10.2	17.1	24.9	33.8	43.4	53.9	65.1	76.9	89.4
.48	.878	4.75	10.3	17.1	25.0	33.9	43.6	54.0	65.2	77.0	89.6
.49	.905	4.80	10.4	17.2	25.1	34.0	43 6	54.1	65.3	77.2	89.7

Table III.—Continued.

Head H, feet.	0	1	2	3	4	5	6	7	8	9	10
0.50	0.934	4.85	10.4	17.3	25.2	34.0	43.7	54.2	65.4	77.3	89.8
.51	.961	4.90	10.5	17.4	25.3	34.1	43.8	54.3	65.5	77.4	90.0
.52	.990	4.95	10.6	17.4	25.4	34.2	44.0	54.4	65.6	77.5	90.1
.53	1.02	5.00	10.6	17.5	25.4	34.3	44.1	54.6	65.8	77.7	90.2
.54	1.05	5.04	10.7	17.6	25.5	34.4	44.2	54.7	65.9	77.8	90.3
.55	1.08	5.09	10.8	17.7	25.6	34.5	44.3	54.8	66.0	77.9	90.5
.56	1.11	5.14	10.8	17.7	25.7	34.6	44.4	54.9	66.1	78.0	90.6
.57	1.14	5.19	10.9	17.8	25.8	34.7	44.5	55.0	66.2	78.2	90.7
.58	1.17	5.24	10.9	17.9	25.9	34.8	44.6	55.1	66.3	78.3	90.8
.59	1.20	5.29	11.0	18.0	26.0	34.9	44.7	55.2	66.5	73.4	91.0
0.60	1.23	5.34	11.1	18.0	26.0	35.0	44.8	55.3	66.6	78.5	91.1
.61	1.26	5.39	11.1	18.1	26.1	35.1	44.9	55.4	66.7	78.6	91.2
.62	1.29	5.44	11.2	18.2	26.2	35.2	45.0	55.5	66.8	78.8	91.4
.63	1.32	5.49	11.2	18.2	26.3	35.3	45.1	55.6	66.9	78.9	91.5
.64	1.35	5.54	11.3	18.3	26.4	35.4	45.2	55.7	67.0	79.0	91.6
.65	1.38	5.60	11.4	18.4	26.5	35.4	45.3	55.9	67.2	79.1	91.8
.66	1.42	5.65	11.4	18.5	26.6	35.5	45.4	56.0	67.3	79.3	91.9
.67	1.45	5.70	11.5	18.6	26.6	35.6	45.5	56.1	67.4	79.4	92.0
.68	1.48	5.75	11.6	18.6	26.7	35.7	45.6	56.2	67.5	79.5	92.1
.69	1.51	5.80	11.6	18.7	26.8	35.8	45.7	56.3	67.6	79.6	92.3
0.70	1.55	5.85	11.7	18.8	26.9	35.9	45.8	56.4	67.7	79.8	92.4
.71	1.58	5.90	11.8	18.9	27.0	36.0	45.9	56.5	67.9	79.9	92.5
.72	1.61	5.96	11.8	18.9	27.1	36.1	46.0	56.6	68.0	80.0	92.7
.73	1.65	6.01	11.9	19.0	27.2	36.2	46.1	56.7	68.1	80.1	92.8
.74	1.68	6.06	12.0	19.1	27.2	36.3	46.2	56.8	68.2	80.2	92.9
.75	1.71	6.11	12.0	19.2	27.3	36.4	46.3	57.0	68.3	80.4	93.0
.76	1.75	6.16	12.1	19.2	27.4	36.5	46.4	57.1	68.4	80.5	93.2
.77	1.78	6.22	12.2	19.3	27.5	36.6	46.5	57.2	68.6	80.6	93.3
.78	1.82	6.27	12.2	19.4	27.6	36.7	46.6	57.3	68.7	80.7	93.4
.79	1.85	6.32	12.3	19.5	27.7	36.8	46.7	57.4	68.8	80.9	93.6
0.80	1.89	6.38	12.4	19.6	27.8	36.9	46.8	57.5	68.9	81.0	93.7
.81	1.92	6.43	12.4	19.6	27.8	37.0	46.9	57.6	69.0	81.1	93.8
.82	1.96	6.48	12.5	19.7	27.9	37.1	47.0	57.7	69.2	81.2	94.0
.83	2.00	6.54	12.6	19.8	28.0	37.2	47.1	57.8	69.3	81.4	94.1
.84	2.03	6.59	12.6	19.9	28.1	37.3	47.2	58.0	69.4	81.5	94.2
.85	2.07	6.64	12.7	19.9	28.2	37.4	47.3	58.1	69.5	81.6	94.4
.86	2.10	6.70	12.8	20.0	28.3	37.4	47.4	58.2	69.6	81.7	94.5
.87	2.14	6.75	12.8	20.1	28.4	37.5	47.5	58.3	69.7	81.9	94.6
.88	2.18	6.80	12.9	20.2	28.5	37.6	47.6	58.4	69.9	82.0	94.7
.89	2.22	6.86	13.0	20.2	28.5	37.7	47.7	58.5	70.0	82.1	94.9
0.90	2.25	6.91	13.0	20.3	28.6	37.8	47.8	58.6	70.1	82.2	95.0
.91	2.29	6.97	13.1	20.4	28.7	37.9	48.0	58.7	70.2	82.4	95.1
.92	2.33	7.02	13.2	20.5	28.8	38.0	48.1	58.8	70.3	82.5	95.3
.93	2.37	7.08	13.2	20.6	28.9	38.1	48.2	59.0	70.4	82.6	95.4
.94	2.41	7.13	13.3	20.6	29.0	38.2	48.3	59.1	70.6	82.7	95.5
.95	2.44	7.19	13.4	20.7	29.1	38.3	48.4	59.2	70.7	82.9	95.6
.96	2.48	7.24	13.4	20.8	29.2	38.4	48.5	59.3	70.8	83.0	95.8
.97	2.52	7.30	13.5	20.9	29.3	38.5	48.6	59.4	70.9	83.1	95.9
.98	2.56	7.36	13.6	21.0	29.3	38.6	48.7	59.5	71.0	83.2	96.0
.99	2.60	7.41	13.6	21.0	29.4	38.7	48.8	59.6	71.2	83.4	96.2
1.00	2.64	7.47	13.7	21.1	29.5	38.8	48.9	59.7	71.3	83.5	96.3

NOTE:—This table is applicable for use with broad-crested weirs exceeding 2 feet of crest width d for heads from 0.5 foot up to 1.5 or 2 times the breadth of weir crest.

DETERMINATION OF DISCHARGE OVER VARIOUS TYPES OF BROAD-CRESTED WEIRS.

From the weir experiments at the Cornell Hydraulic Laboratory, as outlined in United States Geological Survey Water-Supply Paper No. 200, Mr. E. C. Murphy has derived coefficients to be used in connection with a discharge table computed by Bazin's formula for sharp-crested weirs for determining the discharge over certain types of broad-crested weirs. Table IV gives the discharge per foot of length of crest by Bazin's formula for weirs having a height varying from 2 to 30 feet, and tables V, VI, and VII give the multipliers to be used with this table to give the discharge over broad-crested weirs. Example: Suppose the discharge is to be computed over a rectangular weir that is 10 feet long, 12 feet high, 6 feet crest width, and has an observed head of 2.4 feet.

Table IV shows that for a height (p) of 12 feet and a head (H) of 2.4, the discharge (Q) is 12.42 second-feet. Table V shows that for a height (p) of 12 feet, a crest width (c) of 6 feet, and head (H) of 2.4 feet the multiplier is 0.797. Hence, the discharge is $12.42 \times 0.797 \times 10 = 99.0$ second-feet.

Table IV.—Discharge in second-feet per foot of crest over sharp-crested rectangular weirs without end contractions.[a]

$$\text{Formula: } Q = \left(0.405 + \frac{.00984}{H}\right)\left(1 + 0.55\frac{H^2}{(p+H)^2}\right) lH\sqrt{2gH}.$$

[H=head, in feet, P=height of weir, in feet].

H \ p	2	4	6	8	10	20	30
0.1	0.13	0.13	0.13	0.13	0.13	0.13	0.13
.2	.33	.33	.33	.33	.33	.33	.33
.3	.58	.58	.58	.58	.58	.58	.58
.4	.88	.88	.87	.87	.87	.87	.87
.5	1.23	1.21	1.21	1.21	1.21	1.20	1.20
.6	1.62	1.59	1.58	1.58	1.57	1.57	1.57
.7	2.04	1.99	1.98	1.98	1.97	1.97	1.97
.8	2.50	2.43	2.41	2.41	2.40	2.40	2.40
.9	3.00	2.90	2.88	2.86	2.86	2.85	2.85
1.0	3.53	3.40	3.36	3.35	3.34	3.33	3.33
1.1	4.10	3.93	3.88	3.86	3.85	3.84	3.83
1.2	4.69	4.48	4.42	4.40	4.38	4.36	4.36
1.3	5.32	5.07	4.99	4.96	4.94	4.92	4.91
1.4	5.99	5.68	5.58	5.55	5.52	5.49	5.48

[a] This table should not be used where water on the downstream side of the weir is above the level of the crest, nor unless air circulates freely between the overfalling sheet and the downstream face of the weir. If a vacuum forms under the falling sheet the discharge may be 5 per cent greater than given in this table.

Table IV.—Continued.

H \ p	2	4	6	8	10	20	30
1.5	6.69	6.30	6.20	6.16	6.13	6.08	6.07
1.6	7.40	6.97	6.84	6.78	6.75	6.69	6.68
1.7	8.15	7.66	7.50	7.43	7.39	7.33	7.31
1.8	8.93	8.37	8.18	8.09	8.05	7.98	7.96
1.9	9.74	9.11	8.89	8.79	8.74	8.65	8.63
2.0	10.58	9.87	9.62	9.51	9.44	9.34	9.32
2.1	11.44	10.65	10.37	10.24	10.17	10.05	10.02
2.2	12.33	11.46	11.14	10.99	10.91	10.78	10.75
2.3	13.25	12.29	11.93	11.77	11.67	11.52	11.48
2.4	14.20	13.15	12.75	12.56	12.45	12.28	12.24
2.5	15.18	14.03	13.59	13.37	13.25	13.06	13.02
2.6	16.17	14.92	14.44	14.20	14.07	13.85	13.80
2.7	17.19	15.84	15.31	15.05	14.90	14.65	14.60
2.8	18.23	16.79	16.21	15.92	15.76	15.48	15.42
2.9	19.29	17.75	17.12	16.81	16.63	16.32	16.25
3.0	20.38	18.74	18.06	17.71	17.52	17.18	17.10
3.1	21.50	19.74	19.01	18.64	18.42	18.05	17.96
3.2	22.64	20.77	19.98	19.58	19.34	18.93	18.83
3.3	23.80	21.82	20.98	20.54	20.28	19.83	19.72
3.4	24.98	22.89	21.99	21.52	21.24	20.75	20.63
3.5	26.20	23.98	23.01	22.51	22.22	21.69	21.55
3.6	27.42	25.09	24.06	23.52	23.20	22.62	22.48
3.7	28.67	26.23	25.13	24.55	24.21	23.58	23.43
3.8	29.94	27.38	26.22	25.60	25.23	24.56	24.39
3.9	31.23	28.55	27.32	26.66	26.27	25.54	25.37
4.0	32.54	29.74	28.45	27.74	27.32	26.55	26.35
4.1	33.87	30.96	29.59	28.84	28.39	27.56	27.34
4.2	35.22	32.18	30.75	29.96	29.48	28.59	28.35
4.3	36.59	33.43	31.92	31.09	30.58	29.63	29.38
4.4	37.99	34.70	33.12	32.24	31.70	30.68	30.42
4.5	39.40	35.98	34.33	33.40	32.83	31.74	31.47
4.6	40.83	37.29	35.56	34.58	33.98	32.82	32.53
4.7	42.28	38.61	36.80	35.78	35.14	33.92	33.61
4.8	43.75	39.96	38.07	37.00	36.32	35.04	34.70
4.9	45.23	41.32	39.35	38.23	37.52	36.17	35.80
5.0	46.73	42.69	40.65	39.48	38.74	37.21	36.91
5.1	48.25	44.09	41.96	40.73	39.97	38.45	38.03
5.2	49.79	45.50	43.29	42.01	41.20	39.61	39.17
5.3	51.36	46.93	44.64	43.30	42.45	40.78	40.31
5.4	52.94	48.38	46.00	44.60	43.71	41.96	41.47
5.5	54.54	49.85	47.38	45.93	45.00	43.16	42.64
5.6	56.15	51.34	48.79	47.27	46.31	44.38	43.83
5.7	57.78	52.83	50.19	48.62	47.62	45.60	45.02
5.8	59.42	54.34	51.62	49.99	48.94	46.83	46.22
5.9	61.09	55.88	53.07	51.38	50.29	48.08	47.44
6.0	62.77	57.43	54.53	52.78	51.64	49.34	48.67
6.1	64.46	59.00	56.00	54.20	53.02	50.61	49.91
6.2	66.18	60.58	57.50	55.63	54.40	51.90	51.16
6.3	67.91	62.18	59.01	57.07	55.80	53.20	52.42
6.4	69.65	63.79	60.53	58.53	57.22	54.50	53.70

Table IV.—Continued.

H \ p	2	4	6	8	10	20	30
6.5	71.42	65.42	62.07	60.01	58.65	55.82	54.98
6.6	73.19	67.07	63.63	61.50	60.09	57.16	56.27
6.7	74.99	68.74	65.20	63.00	61.55	58.50	57.58
6.8	76.80	70.42	66.78	64.53	63.02	59.96	58.90
6.9	78.62	72.11	68.38	66.06	64.50	61.23	60.22
7.0	80.46	73.82	70.00	67.60	66.00	62.61	61.56
7.1	82.32	75.55	71.63	69.17	67.52	64.00	62.91
7.2	84.18	77.29	73.28	70.74	69.04	65.40	64.27
7.3	86.07	79.04	74.94	72.34	70.58	66.81	65.64
7.4	87.97	80.81	76.61	73.94	72.14	68.24	67.02
7.5	89.89	82.60	78.30	75.56	73.70	69.68	68.41
7.6	91.82	84.40	80.01	77.19	75.28	71.13	69.81
7.7	93.76	86.22	81.73	78.84	76.88	72.59	71.23
7.8	95.72	88.05	83.46	80.50	78.48	74.06	72.65
7.9	97.70	89.90	85.21	82.18	80.11	75.55	74.09
8.0	99.68	91.75	86.97	83.87	81.74	77.04	75.53
8.1	101.69	93.63	88.75	85.57	83.39	78.55	76.98
8.2	103.70	95.51	90.54	87.29	85.25	80.06	78.44
8.3	105.73	97.42	92.34	89.02	86.72	81.59	79.92
8.4	107.78	99.34	94.16	90.76	88.41	83.13	81.40
8.5	109.84	101.27	96.00	92.52	90.11	84.69	82.90
8.6	111.91	103.21	97.84	94.29	91.82	86.25	84.41
8.7	113.99	105.17	99.70	96.07	93.55	87.82	85.92
8.8	116.09	107.14	101.57	97.87	95.28	89.40	87.44
8.9	118.20	109.13	103.46	99.68	97.04	91.00	88.98
9.0	120.33	111.13	105.36	101.50	98.80	92.61	90.52
9.1	122.47	113.15	107.28	103.34	100.58	94.23	92.08
9.2	124.62	115.18	109.21	105.19	102.37	95.86	93.65
9.3	126.79	117.22	111.15	107.06	104.17	97.49	95.22
9.4	128.97	119.27	113.10	108.93	105.99	99.14	96.80
9.5	131.16	121.34	115.07	110.82	107.82	100.80	98.40
9.6	133.36	123.42	117.05	112.72	109.65	102.48	100.00
9.7	135.58	125.51	119.04	114.64	111.50	104.16	101.62
9.8	137.82	127.63	121.05	116.57	113.37	105.85	103.25
9.9	140.06	129.74	123.07	118.51	115.25	107.56	104.88
10.0	142.31	131.87	125.10	120.46	117.14	109.27	106.52

Table V.—Multipliers to be used in connection with Table IV to obtain the discharge over broad-crested weirs of rectangular cross-section of type a, fig. 39.

[p=Height of weir; c=width of crest; H=observed head, all in feet.]

H \ p.. / c..	4.6 2.6	4.6 6.6	11.25 .48	11.25 .93	11.25 1.65	11.25 3.17	11.25 5.88	11.25 8.98	11.25 12.24	11.25 16.30
0.5	....	....	0.821	0.792	0.806	0.792	0.799	0.801	0.786	0.790
1.0	0.765	0.708	.997	.899	.808	.795	.791	.794	.815	.790
1.5	.789	.709	1.00	.982	.878	.796	.796	.793	.814	.792
2.0	.814	.710	1.00	1.00	.906	.815	.797	.792	.797	.793
2.5	.835	.711	1.00	1.00	.985	.844	.797	.790	.796	.793
3.0	.857	.711	1.00	1.00	1.00	.870	.797	.788	.794	.791
3.5	.878	.712	1.00	1.00	1.00	.90	.812	.787	.794	.791
4.0	.899	.714	1.00	1.00	1.00	.93	.834	.786	.792	.789
5.0	.940	.716	1.00	1.00	1.00	.97	(a)	.78	.79	.78
6.0	.986	.718	1.00	1.00	1.00	.98	(a)	.78	.78	.78
7.0	....	....	1.00	1.00	1.00	(a)	(a)	.77	.78	.77
8.0	....	....	1.00	1.00	1.00	(a)	(a)	.77	.77	.77
9.0	....	....	1.00	1.00	1.00	(a)	(a)	.77	.77	.77
10.0	....	....	1.00	1.00	1.00	(a)	(a)	.77	.77	.77

[a] Value doubtful.

Table VI.—Multipliers to be used in connection with Table IV to obtain the discharge over broad-crested weirs of trapezoidal cross-section of types b and c, fig. 39.

[p=Height of weir, in feet; c=width of crest, in feet; s=upstream slope; s'=downstream slope; H=observed head, in feet.]

	Type b, fig. 39.							Type c, fig. 39.	
p	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.65	11.25
c	.33	.66	.66	.66	.66	.33	.66	7.00	6.00
s	2:1	2:1	3:1	4:1	5:1	2:1	2:1	4.67:1	6:1
s'	0	0	0	0	0	5:1	2:1	....	....
H									
1.0........	1.137	1.048	1.066	1.039	1.009	1.095	1.071	1.042	1.060
1.5........	1.131	1.068	1.066	1.039	1.009	1.071	1.066	1.033	1.069
2.0........	1.120	1.080	1.061	1.033	1.005	1.044	1.053	1.024	1.054
2.5........	1.106	1.085	1.052	1.026	.997	1.024	1.047	1.012	1.012
3.0........	1.094	1.088	1.047	1.020	.991	1.009	1.047	.995	.985
3.5........	1.085	1.087	1.043	1.017	.988	1.003	1.050	.983	.979
4.0........	1.072	1.084	1.038	1.012	.984	1.014	1.052	.977	.976
4.5........	1.064	1.081	1.035	1.009	.980	1.023	1.055	.974	.973
5.0........	....	....	....	....	....	....	....	.97	.97
6.0........	....	....	....	....	....	....	....	.97	.96
7.0........	....	....	....	....	....	....	....	.97	.96
8.0........	....	....	....	....	....	....	....	.96	.95
9.0........	....	....	....	....	....	....	....	.96	.95
10.0........	....	....	....	....	....	....	....	.96	.95

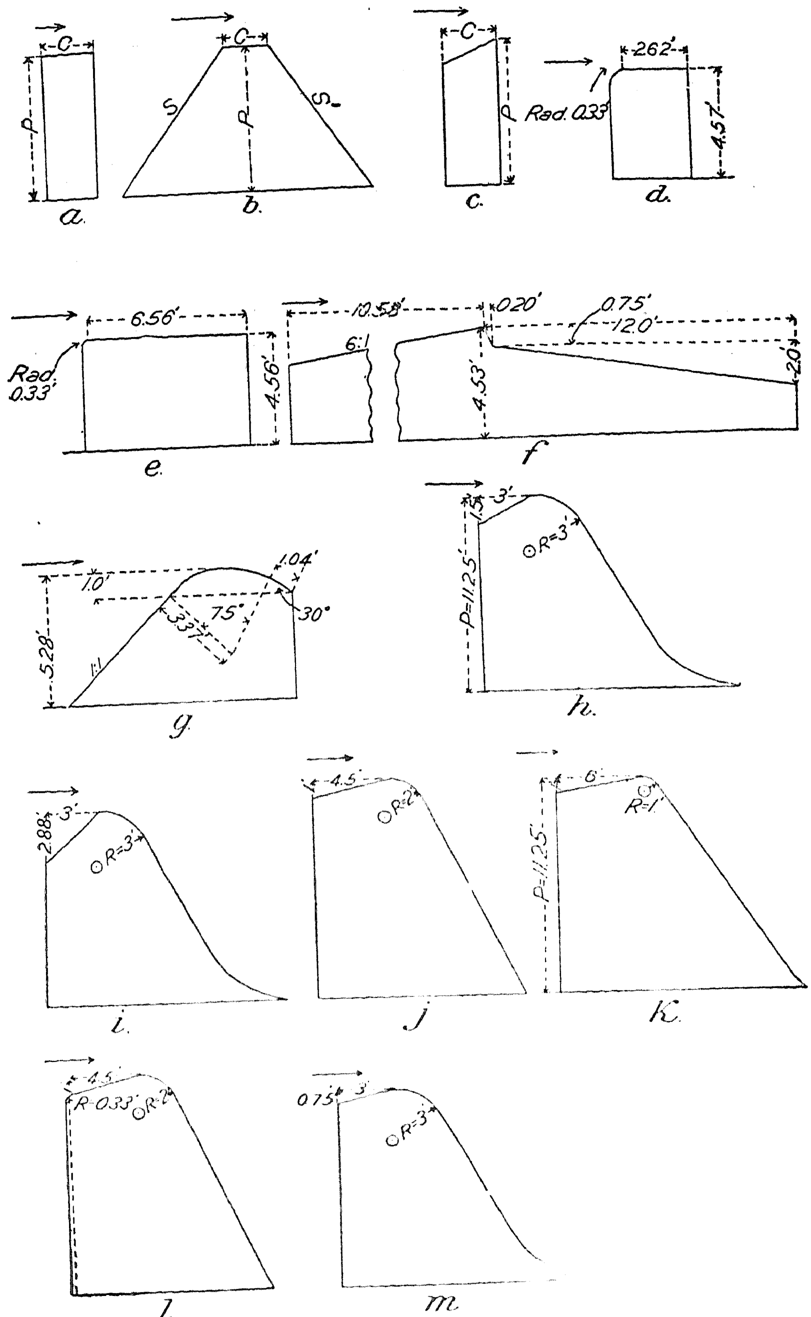

FIG. 39. —Types of Weirs referred to in Tables V, VI, and VII.

Table VII.—Multipliers to be used in connection with Table IV to obtain the discharge over broad-crested weirs of compound cross-section of types d to m inclusive, fig. 39.

[p=Height of weir, in feet; H=observed head, in feet.]

p	4.57	4.56	4.53	5.28	11.25	11.25	11.25	11.25	11.25	11.25
Type, fig. 38	d	e	f	g	h	i	j	k	l	m
H										
0.5	....	....	....	....	0.941	0.924	0.933	0.962	0.971	0.947
1.0	0.842	0.836	0.929	0.976	1.039	1.033	.988	1.045	1.033	1.000
1.5	.866	.834	.950	.979	1.087	1.093	1.018	1.066	1.042	1.036
2.0	.888	.831	.953	.988	1.109	1.133	1.033	1.063	1.035	1.063
2.5	.906	.826	.947	1.000	1.118	1.153	1.045	1.020	1.033	1.085
3.0	.927	.822	.942	1.016	1.120	1.163	1.054	.997	1.045	1.096
3.5	.945	.817	.936	1.032	1.127	1.169	1.060	.994	1.054	1.108
4.0	.965	.812	.931	1.044	1.123	1.165	1.060	.991	1.057	1.110
5.0	1.00	.80	.92	1.05	1.11	1.16	1.05	.98	1.05	1.10
6.0	....	....	....	....	1.11	1.15	1.04	.98	1.04	1.10
7.0	....	....	....	....	1.10	1.14	1.04	.97	1.04	1.09
8.0	....	....	....	....	1.10	1.14	1.04	.97	1.03	1.09
9.0	....	....	....	....	1.09	1.14	1.03	.97	1.03	1.08
10.0	....	....	....	....	1.09	1.13	1.03	.97	1.03	1.08

Table VIII.—Three-halves powers for numbers from 0 to 12.

Units. / Hundredths.	0	1	2	3	4	5	6	7	8	9	10	11
0.00	0.0000	1.0000	2.8284	5.1962	8.0000	11.1803	14.6969	18.5203	22.6274	27.0000	31.6228	36.4829
.01	.0010	1.0150	2.8497	5.2222	8.0300	11.2139	14.7337	18.5600	22.6699	27.0450	31.6702	36.5326
.02	.0028	1.0302	2.8710	5.2482	8.0601	11.2475	14.7705	18.5997	22.7123	27.0890	31.7177	36.5824
.03	.0052	1.0453	2.8923	5.2743	8.0902	11.2811	14.8073	18.6394	22.7548	27.1351	31.7652	36.6322
.04	.0080	1.0606	2.9137	5.3004	8.1203	11.3148	14.8442	18.6792	22.7973	27.1802	31.8127	36.6820
.05	.0112	1.0759	2.9352	5.3266	8.1505	11.3485	14.8810	18.7190	22.8399	27.2253	31.8602	36.7319
.06	.0147	1.0913	2.9567	5.3528	8.1807	11.3822	14.9179	18.7589	22.8825	27.2705	31.9078	36.7818
.07	.0185	1.1068	2.9782	5.3791	8.2109	11.4160	14.9549	18.7988	22.9251	27.3156	31.9554	36.8317
.08	.0226	1.1224	2.9998	5.4054	8.2412	11.4497	14.9919	18.8387	22.9677	27.3608	32.0030	36.8816
.09	.0270	1.1380	3.0215	5.4317	8.2715	11.4836	15.0289	18.8786	23.0103	27.4060	32.0506	36.9315
0.10	0.0316	1.1537	3.0432	5.4581	8.3019	11.5174	15.0659	18.9185	23.0530	27.4512	32.0983	36.9815
.11	.0365	1.1695	3.0650	5.4845	8.3323	11.5513	15.1030	18.9585	23.0957	27.4965	32.1460	37.0315
.12	.0416	1.1853	3.0868	5.5110	8.3627	11.5852	15.1400	18.9985	23.1384	27.5418	32.1937	37.0815
.13	.0469	1.2012	3.1086	5.5375	8.3932	11.6192	15.1772	19.0386	23.1812	27.5871	32.2414	37.1315
.14	.0524	1.2172	3.1306	5.5641	8.4237	11.6532	15.2143	19.0786	23.2240	27.6324	32.2892	37.1816
.15	.0581	1.2332	3.1525	5.5907	8.4542	11.6872	15.2515	19.1187	23.2668	27.6778	32.3370	37.2317
.16	.0640	1.2494	3.1745	5.6173	8.4848	11.7213	15.2887	19.1589	23.3096	27.7232	32.3848	37.2817
.17	.0701	1.2656	3.1966	5.6440	8.5154	11.7554	15.3260	19.1990	23.3525	27.7686	32.4326	37.3319
.18	.0764	1.2818	3.2187	5.6708	8.5460	11.7895	15.3632	19.2392	23.3954	27.8140	32.4804	37.3820
.19	.0828	1.2981	3.2409	5.6975	8.5767	11.8236	15.4005	19.2794	23.4383	27.8595	32.5283	37.4322

Table VIII.—Continued.

Hundredths \ Units	0	1	2	3	4	5	6	7	8	9	10	11
0.20	0.0894	1.3145	3.2631	5.7243	8.6074	11.8578	15.4379	19.3196	23.4812	27.9050	32.5762	37.4824
.21	.0962	1.3310	3.2854	5.7512	8.6382	11.8920	15.4752	19.3599	23.5242	27.9514	32.6241	37.5326
.22	.1032	1.3475	3.3077	5.7781	8.6690	11.9263	15.5126	19.4002	23.5672	27.9960	32.6720	37.5828
.23	.1103	1.3641	3.3301	5.8050	8.6998	11.9606	15.5501	19.4405	23.6102	28.0416	32.7200	37.6331
.24	.1176	1.3808	3.3525	5.8320	8.7307	11.9949	15.5866	19.4808	23.6533	28.0872	32.7680	37.6833
.25	.1250	1.3975	3.3750	5.8590	8.7616	12.0293	15.6250	19.5212	23.6963	28.1328	32.8160	37.7336
.26	.1326	1.4144	3.3975	5.8861	8.7925	12.0636	15.6616	19.5576	23.7394	28.1784	32.8640	37.7840
.27	.1403	1.4312	3.4201	5.9132	8.8235	12.0981	15.7001	19.6021	23.7825	28.2241	32.9121	37.8343
.28	.1482	1.4482	3.4427	5.9403	8.8545	12.1325	15.7376	19.6425	23.8257	28.2698	32.9600	37.8847
.29	.1562	1.4652	3.4654	5.9675	8.8856	12.1670	15.7752	19.6830	23.8689	28.3155	33.0083	37.9351
0.30	0.1643	1.4822	3.4881	5.9947	8.9167	12.2015	15.8129	19.7235	23.9121	28.3612	33.0564	37.9855
.31	.1726	1.4994	3.5109	6.0220	8.9478	12.2361	15.8505	19.7641	23.9553	28.4069	33.1046	38.0359
.32	.1810	1.5166	3.5337	6.0493	8.9790	12.2706	15.8882	19.8046	23.9986	28.4527	33.1527	38.0864
.33	.1896	1.5338	3.5566	6.0767	9.0102	12.3053	15.9260	19.8452	24.0418	28.4985	33.2009	38.1369
.34	.1983	1.5512	3.5795	6.1041	9.0414	12.3399	15.9637	19.8858	24.0851	28.5444	33.2492	38.1874
.35	.2071	1.5686	3.6025	6.1315	9.0726	12.3746	16.0015	19.9265	24.1285	28.5902	33.2974	38.2379
.36	.2160	1.5860	3.6255	6.1590	9.1040	12.4093	16.0393	19.9672	24.1718	28.6361	33.3457	38.2884
.37	.2251	1.6035	3.6486	6.1865	9.1353	12.4440	16.0772	20.0079	24.2152	28.6820	33.3940	38.3390
.38	.2342	1.6211	3.6717	6.2141	9.1667	12.4788	16.1150	20.0486	24.2586	28.7279	33.4423	38.3896
.39	.2436	1.6388	3.6949	6.2417	9.1981	12.5136	16.1529	20.0894	24.3021	28.7739	33.4906	38.4402
0.40	0.2530	1.6565	3.7181	6.2693	9.2295	12.5485	16.1909	20.1302	24.3455	28.8199	33.5390	38.4908
.41	.2625	1.6743	3.7413	6.2970	9.2610	12.5833	16.2288	20.1710	24.3890	28.8659	33.5874	38.5415
.42	.2722	1.6921	3.7646	6.3247	9.2925	12.6182	16.2668	20.2118	24.4325	28.9119	33.6358	38.5922
.43	.282	1.7100	3.7880	6.3525	9.3241	12.6532	16.3048	20.2527	24.4761	28.9579	33.6842	38.6429
.44	.2919	1.7280	3.8114	6.3803	9.3557	12.6882	16.3429	20.2936	24.5196	29.0040	33.7327	38.6936
.45	.3019	1.7460	3.8349	6.4081	9.3873	12.7232	16.3810	20.3345	24.5632	29.0501	33.7811	38.7443
.46	.3120	1.7641	3.8584	6.4360	9.[illegible]189	12.7582	16.4191	20.3755	24.6068	29.0962	33.8297	38.7951
.47	.3222	1.7823	3.8819	6.4639	9.4506	12.7933	16.4572	20.4165	24.6505	29.1424	33.8782	38.8459
.48	.3325	1.8005	3.9055	6.4919	9.4824	12.8284	16.4954	20.4575	24.6941	29.1885	33.9267	38.8967
.49	.3430	1.8188	3.9292	6.5199	9.5141	12.8635	16.5336	20.4985	24.7378	29.2347	33.9753	38.9475
0.50	0.3536	1.8371	3.9529	6.5479	9.5459	12.8986	16.5718	20.5396	24.7815	29.2810	34.0239	38.9984
.51	.3642	1.8555	3.9766	6.5760	9.5778	12.9338	16.6101	20.5807	24.8253	29.3272	34.0725	39.0493
.52	.3750	1.8740	4.0004	6.6041	9.6097	12.9691	16.6484	20.6218	24.8691	29.3735	34.1211	39.1002
.53	.3858	1.8925	4.0242	6.6323	9.6416	13.0043	16.6867	20.6630	24.9129	29.4198	34.1698	39.1511
.54	.3968	1.9111	4.0481	6.6605	9.6735	13.0396	16.7250	20.7041	24.9567	29.4661	34.2185	39.2020
.55	.4079	1.9297	4.0720	6.6887	9.7055	13.0749	16.7634	20.7453	25.0005	29.5124	34.2672	39.2530
.56	.4191	1.9484	4.0960	6.7170	9.7375	13.1103	16.8018	20.7866	25.0444	29.5588	34.3159	39.3040
.57	.4303	1.9672	4.1200	6.7453	9.7695	13.1457	16.8402	20.8278	25.0883	29.6052	34.3647	39.3550
.58	.4417	1.9860	4.1441	6.7737	9.8016	13.1811	16.8787	20.8691	25.1322	29.6516	34.4135	39.4060
.59	.4532	2.0049	4.1682	6.8021	9.8337	13.2165	16.9172	20.9104	25.1762	29.6980	34.4623	39.4571
0.60	0.4648	2.0238	4.1924	6.8305	9.8659	13.2520	16.9557	20.9518	25.2202	29.7445	34.5111	39.5082
.61	.4764	2.0429	4.2166	6.8590	9.8981	13.2875	16.9943	20.9931	25.2642	29.7910	34.5599	39.5593
.62	.4882	2.0619	4.2408	6.8875	9.9303	13.3231	17.0328	21.0345	25.3082	29.8375	34.6088	39.6104
.63	.5000	2.0810	4.2651	6.9161	9.9626	13.3587	17.0714	21.0759	25.[illegible]22	29.8841	34.6577	39.6615
.64	.5120	2.1002	4.2895	6.9447	9.9949	13.3943	17.1101	21.1174	25.3963	29.9306	34.7066	39.7127
.65	.5240	2.1195	4.3139	6.9733	10.0272	13.4299	17.1488	21.1589	25.4404	29.9772	34.7557	39.7639
.66	.5362	2.1388	4.3383	7.0020	10.0596	13.4656	17.1874	21.2004	25.4845	30.0238	34.8045	39.8151
.67	.5484	2.1581	4.3628	7.0307	10.0920	13.5013	17.2172	21.2419	25.5287	30.0704	34.8535	39.8663
.68	.5607	2.1775	4.3874	7.0595	10.1244	13.5370	17.2649	21.2834	25.5729	30.1171	34.9025	39.9176
.69	.5732	2.1970	4.4119	7.0883	10.1569	13.5728	17.3037	21.3250	25.6171	30.1638	34.9516	39.2689

Table VIII.—Continued.

Hundredths. \ Units.	0	1	2	3	4	5	6	7	8	9	10	11
0.70	0.5857	2.2165	4.4366	7.1171	10.1894	13.6086	17.3425	21.3666	25.6613	30.2105	35.0006	40.0202
.71	.5983	2.2361	4.4612	7.1460	10.2214	13.6444	17.3814	21.4083	25.7056	30.2572	35.0497	40.0715
.72	.6109	2.2558	4.4859	7.1749	10.2545	13.6803	17.4202	21.4499	25.7499	30.3040	35.0988	40.1228
.73	.6237	2.2755	4.5107	7.2038	10.2871	13.7161	17.4591	21.4916	25.7942	30.3507	35.1479	40.1742
.74	.6366	2.2952	4.5355	7.2328	10.3197	13.7521	17.4981	21.5333	25.8395	30.3975	35.1971	40.2256
.75	.6495	2.3150	4.5604	7.2618	10.3524	13.7880	17.5370	21.5751	25.8828	30.4444	35.2462	40.2770
.76	.6626	2.3349	4.5853	7.2909	10.3851	13.8240	17.5760	21.6169	25.9272	30.4912	35.2954	40.3284
.77	.6757	2.3548	4.6102	7.3200	10.4178	13.8600	17.6150	21.6587	25.9716	30.5381	35.3446	40.3798
.78	.6889	2.3748	4.6352	7.3492	10.4506	13.8961	17.6541	21.7005	26.0161	30.5850	35.3939	40.4313
.79	.7022	2.3949	4.6602	7.3783	10.4834	13.9321	17.6931	21.7423	26.0605	30.6319	35.4431	40.4828
0.80	0.7155	2.4150	4.6853	7.4076	10.5163	13.9682	17.7322	21.7842	26.1050	30.6789	35.4924	40.5343
.81	.7290	2.4351	4.7104	7.4368	10.5492	14.0044	17.7714	21.8261	26.1495	30.7258	35.5417	40.5859
.82	.7425	2.4553	4.7356	7.4661	10.5812	14.0406	17.8105	21.8681	26.1941	30.7728	35.5911	40.6374
.83	.7562	2.4756	4.7608	7.4955	10.6150	14.0768	17.8507	21.9100	26.2386	30.8198	35.6404	40.6890
.84	.7699	2.4959	4.7861	7.5248	10.6480	14.1130	17.8889	21.9520	26.2832	30.8669	35.6898	40.7406
.85	.7837	2.5163	4.8114	7.5542	10.6810	14.1493	17.9282	21.9940	26.3278	30.9139	35.7392	40.7922
.86	.7975	2.5367	4.8367	7.5837	10.7141	14.1856	17.9674	22.0361	26.3725	30.9610	35.7886	40.8439
.87	.8115	2.5572	4.8621	7.6132	10.7472	14.2219	18.0067	22.0781	26.4171	31.0081	35.8380	40.8955
.88	.8255	2.5777	4.8875	7.6427	10.7803	14.2582	18.0461	22.1202	26.4618	31.0553	35.8875	40.9472
.89	.8396	2.5983	4.9130	7.6723	10.8134	14.2946	18.0854	22.1623	26.5065	31.1024	35.9370	40.9989
0.90	0.8538	2.6190	4.9385	7.7019	10.8466	14.3311	18.1248	22.2045	26.5523	31.1496	35.9865	41.0507
.91	.8681	2.6397	4.9641	7.7315	10.8798	14.3675	18.1642	22.2467	26.5960	31.1968	36.0360	41.1024
.92	.8824	2.6604	4.9897	7.7702	10.9131	14.4040	18.2037	22.2889	26.6408	31.2441	36.0856	41.1542
.93	.8969	2.6812	5.0154	7.7909	10.9464	14.4405	18.2432	22.3311	26.6856	31.2913	36.1352	41.2060
.94	.9114	2.7021	5.0411	7.8207	10.9797	14.4770	18.2827	22.3733	26.7305	31.3386	36.1848	41.2578
.95	.9259	2.7230	5.0668	7.8505	11.0131	14.5136	18.3222	22.4156	26.7753	31.3850	36.2344	41.3097
.96	.9406	2.7440	5.0926	7.8803	11.0464	14.5502	18.3617	22.4579	26.8202	31.4332	36.2841	41.3615
.97	.9553	2.7650	5.1184	7.9102	11.0799	14.5869	18.4013	22.5003	26.8651	31.4806	36.3337	41.4134
.98	.9702	2.7861	5.1443	7.9401	11.1133	14.6235	18.4409	22.5426	26.9100	31.5280	36.3834	41.4653
.99	.9850	2.8072	5.1702	7.9700	11.1468	14.6602	18.4806	22.5850	26.9550	31.5754	36.4331	41.5173
1.00	1.0000	2.8284	5.1962	8.0000	11.1803	14.6969	18.5203	22.6274	27.0000	31.6228	36.4829	41.5692

Table IX.—For converting discharge in second-feet per square mile into run-off in depth in inches over the area.

Second-feet per square mile.	Period in days.				
	1	28	29	30	31
	Inches.	*Inches.*	*Inches.*	*Inches.*	*Inches.*
1	.03719	1.041	1.079	1.116	1.153
2	.07438	2.083	2.157	2.231	[illegible]
3	.11157	3.124	3 · 36	3.347	3.459
4	.14876	4.165	4.314	4.463	4.612
5	.18595	5.207	5.393	5.579	5.764
6	.22314	6.248	6.471	6.694	6.917
7	.26033	7.289	7.550	7.810	8.070
8	.29752	8.331	8.628	8.926	9.223
9	.33471	9.372	9.707	10.041	10.376

NOTE.—For partial month multiply the values for one day by the number of days.

Table X.—For converting discharge in second-feet into run-off in acre-feet.

Second-feet.	Period in days.				
	1	28	29	30	31
	Acre-ft.	*Acre-ft.*	*Acre-ft.*	*Acre-ft.*	*Acre-ft.*
1	1.983	55.54	57.52	59.50	61.49
2	3.967	111.1	115.0	119.0	123.0
3	5.950	166.6	172.6	178.5	184.5
4	7.934	222.1	230.1	238.0	246.0
5	9.917	277.7	287.6	297.5	307.4
6	11.90	333.2	345.1	357.0	368.9
7	13.88	388.8	402.6	416.5	430.4
8	15.87	444.3	460.2	476.0	491.9
9	17.85	499.8	517.7	535.5	553.4

NOTE.—For partial month multiply values for one day by the number of days.

Table XI.—For converting discharge in second-feet per day into run-off in millions of gallons.

1 second-foot, or 7.4805 gallons per second for 1 day, or 86,400 seconds = 646,300 gallons.

Tens.	Units.									
	0	1	2	3	4	5	6	7	8	9
0	....	0.65	1.29	1.94	2.59	3.23	3.88	4.52	5.17	5.82
1	6.46	7.11	7.76	8.40	9.05	9.69	10.34	10.99	11.63	12.28
2	12.93	13.57	14.22	14.87	15.51	16.16	16.80	17.45	18.10	18.74
3	19.39	20.04	20.68	21.33	21.97	22.62	23.27	23.91	24.56	25.21
4	25.85	26.50	27.15	27.79	28.44	29.08	29.73	30.38	31.02	31.67
5	32.32	32.96	33.61	34.25	34.90	35.55	36.19	36.84	37.49	38.13
6	38.78	39.43	40.07	40.72	41.36	42.01	42.66	43.30	43.95	44.60
7	45.24	45.89	46.53	47.18	47.83	48.47	49.12	49.77	50.41	51.06
8	51.71	52.35	53.00	53.64	54.29	54.94	55.58	56.23	56.88	57.52
9	58.17	58.81	59.46	60.11	60.75	61.40	62.05	62.69	63.34	63.99

Table XII.—For converting run-off in millions of gallons into discharge in second-feet per day.

1 million gallons per 24 hours $= \frac{231,000,000}{1,728 \cdot 86,400}$ cubic feet per second, or 1.547 second feet.

Tens.	Units.									
	0	1	2	3	4	5	6	7	8	9
0	....	1.55	3.09	4.64	6.19	7.74	9.28	10.83	12.38	13.93
1	15.47	17.02	18.57	20.11	21.66	23.21	24.76	26.30	27.85	29.40
2	30.94	32.49	34.04	35.59	37.13	38.68	40.23	41.78	43.32	44.87
3	46.42	47.96	49.51	51.06	52.61	54.15	55.70	57.25	58.79	60.34
4	61.89	63.44	64.98	66.53	68.08	69.63	71.17	72.72	74.27	75.81
5	77.36	78.91	80.46	82.00	83.55	85.10	86.64	88.19	89.74	91.29
6	92.83	94.38	95.93	97.48	99.02	100.57	102.12	103.66	105.21	106.76
7	108.31	109.85	111.40	112.95	114.49	116.04	117.59	119.14	120.68	122.23
8	123.78	125.33	126.87	128.42	129.97	131.51	133.06	134.61	136.16	137.70
9	139.25	140.80	142.34	143.89	145.44	146.99	148.53	150.08	151.63	153.18

Table XIII.—For converting run-off in acre-feet into run-off in million gallons.

1 acre-foot = 43,560 cubic feet $= \frac{43,560 \times 1,728}{231}$, or $\frac{75,271,680}{231}$ or 325,850 gallons.

Tens.	Units. 0	1	2	3	4	5	6	7	8	9
0	....	0.326	0.652	0.978	1.303	1.629	1.955	2.281	2.607	2.933
1	3.258	3.584	3.910	4.236	4.562	4.888	5.214	5.540	5.865	6.191
2	6.517	5.843	7.169	7.495	7.820	8.146	8.472	8.798	9.124	9.450
3	9.776	10.101	10.427	10.753	11.079	11.405	11.731	12.056	12.382	12.708
4	13.034	13.360	13.686	14.012	14.337	14.663	14.989	15.315	15.641	15.967
5	16.293	16.618	16.944	17.270	17.596	17.922	18.248	18.574	18.899	19.225
6	19.551	19.877	20.203	20.529	20.854	21.180	21.506	21.832	22.158	22.484
7	22.810	23.135	23.461	23.787	24.113	24.439	24.765	25.090	25.416	25.742
8	26.068	26.394	26.720	27.046	27.372	27.697	28.023	28.349	28.675	29.001
9	29.327	29.652	29.978	30.304	30.630	30.956	31.282	31.608	31.933	32 259

Table XIV.—For converting run-off in million gallons into run-off in acre-feet.

One million United States liquid gallons or 231 million cubic inches = 133,680,555 cubic feet, or $\frac{133,680}{43,560} = 3.0689$ acre-feet.

Tens.	Units. 0	1	2	3	4	5	6	7	8	9
0	....	3.07	6.14	9.21	12.28	15.34	18.41	21.48	24.55	27.62
1	30.69	33.76	36.83	39.90	42.96	46.03	49.10	52.17	55.24	58.31
2	61.38	64.45	67.52	70.58	73.65	76.72	79.79	82.86	85.93	89.00
3	92.07	95.14	98.20	101.27	104.34	107.41	110.48	113.55	116.62	119.69
4	122.76	125.82	128.89	131.96	135.03	138.10	141.17	144.24	147.31	150.38
5	153.44	156.51	159.58	162.65	165.72	168.79	171.86	174.93	178.00	181.06
6	184.13	187.20	190.27	193.34	196.41	199.48	202.55	205.62	208.68	211.75
7	214.82	217.89	220.96	224.03	227.10	230.17	233.24	236.30	239.37	242.44
8	245.51	248.58	251.65	254.72	257.79	260.86	263.92	266.99	270.06	273.13
9	276.20	279.27	282.34	285.41	288.48	291 54	294.61	297.68	300.75	303.82

Table XV.—Values of c for use in the Chezy formula $V = c\sqrt{Rs}$.

Slope.	R.	n .020	n .025	n .030	n .035	n .040	n .045	n .050	n .055	n .060
.0001	3.28	91	73	60	52	46	40	36	33	30
	10	111	92	78	69	62	55	50	46	42
	20	122	102	89	79	71	65	60	55	51
	50	134	114	100	91	83	76	71	67	63
	100	140	121	108	98	91	84	79	74	70
.0002	10	108	89	76	67	60	53	49	45	41
	20	117	98	85	76	68	61	57	53	49
	50	126	108	94	85	78	71	66	62	58
	100	131	113	99	90	83	77	72	68	64
.0004	10	107	88	75	66	59	53	48	44	41
	20	115	96	83	73	66	60	55	51	48
	50	123	104	91	82	75	68	63	59	56
	100	127	108	96	87	80	73	68	64	61
.0010	10	105	87	74	65	58	52	47	44	40
	20	113	94	81	72	65	59	54	50	47
	50	120	101	89	79	72	66	61	57	54
	100	124	105	94	85	77	71	66	62	59
.010	10	105	86	74	65	58	51	47	43	40
	20	112	93	80	71	64	58	53	49	46
	50	119	100	87	78	71	65	60	56	53
	100	122	104	91	82	75	69	65	61	58

NOTE.—For $R = 3.28$ feet, n constant, c is constant for all values of slope. For slopes

Table XVI.—Square roots of numbers ($\sqrt{R}$ $\sqrt{s}$) for use in Kutter's formula. See Pl. VII.

R	$\sqrt{R}$	R	$\sqrt{R}$	R	$\sqrt{R}$	R	$\sqrt{R}$	R	$\sqrt{R}$	s	$\sqrt{s}$
0.05	0.224	3.05	1.746	6.05	2.460	9.05	3.008	20.25	4.500	.00002	.00447
0.10	0.316	3.10	1.761	6.10	2.470	9.10	3.017	20.50	4.528	.000025	.005
0.15	0.387	3.15	1.775	6.15	2.480	9.15	3.025	20.75	4.555	.0000275	.00524
0.20	0.447	3.20	1.789	6.20	2.490	9.20	3.033	21.00	4.583	.00003	.00548
0.25	0.500	3.25	1.803	6.25	2.500	9.25	3.041	21.25	4.610	.000035	.00592
0.30	0.548	3.30	1.817	6.30	2.510	9.30	3.050	21.50	4.637	.00004	.00632
0.35	0.592	3.35	1.830	6.35	2.520	9.35	3.058	21.75	4.664	.000045	.00671
0.40	0.632	3.40	1.844	6.40	2.530	9.40	3.066	22.00	4.690	.00005	.00707
0.45	0.671	3.45	1.857	6.45	2.540	9.45	3.074	22.25	4.717	.00006	.00775
0.50	0.707	3.50	1.871	6.50	2.550	9.50	3.082	22.50	4.743	.00007	.00837
0.55	0.742	3.55	1.884	6.55	2.559	9.55	3.090	22.75	4.770	.00008	.00894
0.60	0.775	3.60	1.897	6.60	2.569	9.60	3.098	23.00	4.796	.00009	.00949
0.65	0.806	3.65	1.910	6.65	2.579	9.65	3.106	23.25	4.822	.0001	.01
0.70	0.837	3.70	1.924	6.70	2.588	9.70	3.114	23.50	4.848	.00012	.0110
0.75	0.866	3.75	1.936	6.75	2.598	9.75	3.122	23.75	4.873	.00014	.0118
0.80	0.894	3.80	1.949	6.80	2.608	9.80	3.130	24.00	4.899	.00016	.0126
0.85	0.922	3.85	1.962	6.85	2.617	9.85	3.138	24.25	4.924	.00018	.0134
0.90	0.949	3.90	1.975	6.90	2.627	9.90	3.146	24.50	4.950	.0002	.0141
0.95	0.975	3.95	1.987	6.95	2.636	9.95	3.154	24.75	4.975	.00025	.0158
1.00	1.000	4.00	2.000	7.00	2.646	10.00	3.162	25.00	5.000	.0003	.0173
1.05	1.025	4.05	2.012	7.05	2.655	10.25	3.202	25.25	5.025	.00035	.0187
1.10	1.049	4.10	2.025	7.10	2.665	10.50	3.240	25.50	5.050	.0004	.02
1.15	1.072	4.15	2.037	7.15	2.674	10.75	3.279	25.75	5.074	.0005	.0224
1.20	1.095	4.20	2.049	7.20	2.683	11.00	3.317	26.00	5.099	.0006	.0245
1.25	1.118	4.25	2.062	7.25	2.693	11.25	3.354	26.25	5.123	.0007	.0265
1.30	1.140	4.30	2.074	7.30	2.702	11.50	3.391	26.50	5.148	.0008	.0283
1.35	1.162	4.35	2.086	7.35	2.711	11.75	3.428	26.75	5.172	.0009	.03
1.40	1.183	4.40	2.098	7.40	2.720	12.00	3.464	27.00	5.196	.001	.0316
1.45	1.204	4.45	2.110	7.45	2.729	12.25	3.500	27.25	5.220	.0012	.0346
1.50	1.225	4.50	2.121	7.50	2.739	12.50	3.536	27.50	5.244	.0014	.0374
1.55	1.245	4.55	2.133	7.55	2.748	12.75	3.571	27.75	5.268	.0016	.04
1.60	1.265	4.60	2.145	7.60	2.757	13.00	3.606	28.00	5.292	.0018	.0424
1.65	1.285	4.65	2.156	7.65	2.766	13.25	3.640	28.25	5.315	.002	.0447
1.70	1.304	4.70	2.168	7.70	2.775	13.50	3.674	28.50	5.339	.003	.0548
1.75	1.323	4.75	2.179	7.75	2.784	13.75	3.708	28.75	5.362	.004	.0632
1.80	1.342	4.80	2.191	7.80	2.793	14.00	3.742	29.00	5.385	.005	.0707
1.85	1.360	4.85	2.202	7.85	2.802	14.25	3.775	29.25	5.408	....	......
1.90	1.378	4.90	2.214	7.90	2.811	14.50	3.808	29.50	5.431	....	......
1.95	1.396	4.95	2.225	7.95	2.820	14.75	3.841	29.75	5.454	....	......
2.00	1.414	5.00	2.236	8.00	2.828	15.00	3.873	30.00	5.477	....	......
2.05	1.432	5.05	2.247	8.05	2.837	15.25	3.905	30.25	5.500	....	......
2.10	1.449	5.10	2.258	8.10	2.846	15.50	3.937	30.50	5.523	....	......
2.15	1.466	5.15	2.269	8.15	2.855	15.75	3.969	30.75	5.545	....	......
2.20	1.483	5.20	2.280	8.20	2.864	16.00	4.000	31.00	5.568	....	......
2.25	1.500	5.25	2.291	8.25	2.872	16.25	4.031	31.25	5.590	....	......
2.30	1.517	5.30	2.302	8.30	2.881	16.50	4.062	31.50	5.612	....	......
2.35	1.533	5.35	2.313	8.35	2.890	16.75	4.093	31.75	5.635	....	......
2.40	1.549	5.40	2.324	8.40	2.898	17.00	4.123	32.00	5.657	....	......
2.45	1.565	5.45	2.335	8.45	2.907	17.25	4.153	32.25	5.679	....	......
2.50	1.581	5.50	2.345	8.50	2.915	17.50	4.183	32.50	5.701	....	......
2.55	1.597	5.55	2.356	8.55	2.924	17.75	4.213	32.75	5.723	....	......
2.60	1.612	5.60	2.366	8.60	2.933	18.00	4.243	33.00	5.745	....	......
2.65	1.628	5.65	2.377	8.65	2.941	18.25	4.272	33.25	5.766	....	......
2.70	1.643	5.70	2.387	8.70	2.950	18.50	4.301	33.50	5.788	....	......
2.75	1.658	5.75	2.398	8.75	2.958	18.75	4.330	33.75	5.809	....	......
2.80	1.673	5.80	2.408	8.80	2.966	19.00	4.359	34.00	5.831	....	......
2.85	1.688	5.85	2.419	8.85	2.975	19.25	4.387	34.25	5.852	....	......
2.90	1.703	5.90	2.429	8.90	2.983	19.50	4.416	34.50	5.874	....	......
2.95	1.718	5.95	2.439	8.95	2.992	19.75	4.444	34.75	5.895	....	......
3.00	1.732	6.00	2.449	9.00	3.000	20.00	4.472	35.00	5.916	....	......

Table XVII.—Convenient equivalents.

1 second-foot equals 40 California miner's inches. (Law of March 23, 1901.)
1 second-foot equals 38.4 Colorado miner's inches.
1 second-foot equals 40 Arizona miner's inches.
1 second-foot equals 7.48 United States gallons per second; equals 448.8 gallons per minute; equals 646,272 gallons for one day.
1 second-foot equals 6.23 British imperial gallons per second.
1 second-foot for one year covers one square mile 1.131 feet deep; 13.57 inches deep.
1 second-foot for one year equals 31,536,000 cubic feet.
1 second-foot equals about 1 acre-inch per hour.
1 second-foot falling 10 feet equals 1.136 horsepower.
100 California miner's inches equal 15.7 United States gallons per second.
100 California miner's inches equal 96.0 Colorado miner's inches.
100 California miner's inches for one day equal 4.96 acre-feet.
100 Colorado miner's inches equal 2.60 second-feet.
100 Colorado miner's inches equal 19.5 United States gallons per second.
100 Colorado miner's inches equal 104 California miner's inches.
100 Colorado miner's inches for one day equal 5.17 acre-feet.
100 United States gallons per minute equal 0.223 second-foot.
100 United States gallons per minute for one day equal 0.442 acre-foot.
1,000,000 United States gallons per day equal 1.55 second-feet.
1,000,000 United States gallons equal 3.07 acre-feet.
1,000,000 cubic feet equal 22.95 acre-feet.
1 acre-foot equals 325,850 gallons.
1 inch deep on 1 square mile equals 2,323,200 cubic feet.
1 inch deep on 1 square mile equals 0.0737 second-foot per year.
1 inch equals 2.54 centimeters.
1 foot equals 0.3048 meter.
1 yard equals 0.9144 meter.
1 mile equals 1.60935 kilometers.
1 mile equals 1,760 yards; equals 5.280 feet; equals 63,360 inches.
1 square yard equals 0.836 square meter.
1 acre equals 0.4047 hectare
1 acre equals 43,560 square feet; equals 4,840 square yards.
1 acre equals 209 feet square, nearly.
1 square mile equals 259 hectares.
1 square mile equals 2.59 square kilometers.
1 cubic foot equals 0.0283 cubic meter.
1 cubic foot equals 7.48 gallons; equals 0.804 bushel.
1 cubic foot of water weighs 62.5 pounds.
1 cubic yard equals 0.7646 cubic meter.
1 gallon equals 3.7854 liters.
1 gallon equals 8.36 pounds of water.
1 gallon equals 231 cubic inches (liquid measure).
1 pound equals 0.4536 kilogram.
1 avoirdupois pound equals 7,000 grains.
1 troy pound equals 5,760 grams.

1 meter equals 39.37 inches. Log. 1.5951654.
1 meter equals 3.280833 feet. Log. 0.5159842.
1 meter equals 1.093611 yards. Log. 0.0388629.
1 kilometer equals 3,281 feet; equals five-eighths mile, nearly.
1 square meter equals 10.764 square feet; equals 1.196 square yards.
1 hectare equals 2.471 acres.
1 cubic meter equals 35.314 cubic feet; equals 1.308 cubic yards.
1 liter equals 1.0567 quarts.
1 gram equals 15.43 grains.
1 kilogram equals 2.2046 pounds.
1 tonneau equals 2,204.6 pounds.
1 foot per second equals 1.097 kilometers per hour.
1 foot per second equals 0.68 mile per hour.
1 cubic meter per minute equals 0.5886 second-foot.
1 atmosphere equals 15 pounds per square inch; equals 1 ton per square foot; equals 1 kilogram per square centimeter.

Acceleration of gravity equals 32.16 feet per second every second.
1 horsepower equals 550 foot-pounds per second.
1 horsepower equals 76.04 kilogram-meters per second.
1 horsepower equals 746 watts.
1 horsepower equals 1 second-foot falling 8.80 feet.
$1\frac{1}{3}$ horsepowers equal about 1 kilowatt.

To calculate water power quickly: $\frac{\text{Sec.-ft.} \times \text{fall in feet}}{11}$ = Net horsepower on water wheel, realizing 80 per cent of the theoretical power.

To change miles to inches on map:

Scale 1:125000, 1 mile = 0.50688 inch.
Scale 1:90000, 1 mile = 0.70400 inch.
Scale 1:62500, 1 mile = 1.01376 inches.
Scale 1:45000, 1 mile = 1.40800 inches.

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